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Nevada Creek
Dam, Helmville,
Montana, Powell
County, Mt 00017

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NEVADA CREEK DAM
HELMVILLE, MONTANA
POWELL COUNTY
MT 00017

prepared for:

HONORABLE TED SCHWINDEN
GOVERNOR, STATE OF MONTANA

MONTANA DEPARTMENT OF NATURAL
RESOURCES AND CONSERVATION
(OWNER-ADMINISTRATOR)

NEVADA CREEK WATER USERS' ASSOCIATION
(OPERATOR)

prepared by:

HKM ASSOCIATES
BILLINGS, MONTANA

JANUARY 1981

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EXECUTIVE SUMMARY

Personnel of HKM Associates, under a contract with the Montana Department of Natural Resources and Conservation (DNRC), and with representation from the DNRC, and the Nevada Creek Water Users' Association, inspected Nevada Creek Dam on September 25, 1979. The inspection and evaluation was performed under the authority of Public Law 92-367. Nevada Creek Dam is located on Nevada Creek, approximately 8.5 miles southeast of Helmville, Powell County, Montana. In terms of stream course length, the dam is located about 9 miles upstream of Helmville. Dam construction was basically completed in December 1938, with some grouting for seepage control in 1940.

This report was compiled from information obtained during the onsite inspection, review of construction plans and project history files, and an analysis of available geologic and hydrologic information. Findings were compared with criteria provided in the U.S. Army Corps of Engineers Recommended Guidelines for Safety Inspection of Dams (Ref. 1).

FINDINGS AND EVALUATION

Nevada Lake stores runoff from a drainage area of 145 square miles. The project is multi-purpose in nature as the water is used for irrigation and recreation, and storage is provided for sediment accumulation and flood water detention. Usable capacity from the bottom of the outlet to the spillway crest is 12,414 acre-feet (AF). Total storage capacity to the existing first overtopping elevation is 17,300 AF. The dam has a structural height of 105 feet and a hydraulic height of 88 feet. On the basis of criteria in the U.S. Army Corps of Engineers Recommended Guidelines for Safety Inspection of Dams (Ref. 1), the project is intermediate in size. Based on a visual inspection and engineering judgement, approximately 6 residences located in the flood plain, roads, bridges, irrigation water delivery systems and miscellaneous utilities could be affected by a sudden breach of the dam. The downstream hazard potential is therefore high (Category 1). However, no dam breach analysis or routing of a dam breach flood was made for the downstream area.

The guidelines recommend that the discharge and/or storage capacity of an intermediate-size, high downstream hazard potential dam be capable of safely handling the Probable Maximum Flood (PMF). The PMF is defined as a flood expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in the region. Routing of the estimated PMF developed for the safety evaluation of Nevada Creek Dam showed that the dam is capable of controlling a flood having ordinates approximately equal to 34 percent of the PMF hydrograph ordinates. The embankment materials are classified as being moderately erodible and

can be expected to remain stable only for a short period of time under an overtopping condition. A flood event greater than 34 percent of the PMF ordinates results in overtopping and failure of the dam.

The embankment structure showed no signs of instability, however, sufficient information is not presently available to permit evaluation of stability. The downstream face was constructed on a 1 vertical (V) on 2 horizontal (H) slope. There is a considerable amount of spring activity in the project area, particularly along the right (north) downstream valley wall and right bank of the return channel. It appears the seepage control system was operating at or near capacity at the time of the field investigation. This is of some concern considering the fact that the reservoir pool was at a relatively low level during the investigation.

A comparison of report findings with inspection guidelines shows Nevada Creek Dam to have insufficient storage and/or discharge capacity to safely handle the recommended spillway design flood (SDF), which is the full PMF. Because the project can not safely handle one-half the PMF, the spillway is considered seriously inadequate and the project is considered unsafe (non-emergency) until deficiencies are corrected.

RECOMMENDATIONS

Immediately develop, implement and test a downstream warning plan, and implement a more active maintenance program. Special immediate attention and rehabilitation efforts must be directed to the spillway because of its deteriorating condition and inadequate hydraulic capacity. Recommended short-term and minor improvements in the spillway include: brush removal in the approach channel, concrete patching where required to maintain structural integrity, rearrangement/replacement of rock riprap in strategic areas of the stilling basin, and providing positive drainage from the basin. It is recommended that maximum reservoir management be employed to prevent large discharges from flowing through the existing spillway until it is repaired or replaced. Inspect the outlet conduit from the intake structure to the valve chamber. Perform minor repair work on the 54-inch emergency and operating valves, the deflector plate on the pivot valve, and the leaky sections of the outlet works conduit. Repair of the outlet works stilling basin should include: rebuilding the riprap, particularly where the mortared section has deteriorated; rehabilitating the mortared, rock riprap section along the right bank which has subsided; stabilizing the right bank by flattening the areas with steep slopes and/or by placing a thick granular section over the fine-grained soils; reshaping the right bank where a slope failure has occurred and replacing the riprap; and and rearranging and/or replacing rock riprap immediately

downstream of the concrete outlet structure to eliminate future scouring and undercutting. Install piezometers in the embankment, abutments, and foundation, and establish a regular monitoring program to evaluate piezometric conditions. Establish reference points along the embankment and at exterior sites to monitor long-term settlements and slope movements. Periodically remove large debris from the upstream face of the dam. Repair the concrete on the upstream face of the control weir. Replace the 18-inch slidegate and frame on the weir structure. Replace and recalibrate the staff gage upstream of the control weir.

Engineering studies are recommended to better determine the downstream hazard and required spillway capacity and to evaluate embankment structural stability. A remedial measures design program is recommended with consideration of the following: facility enhancement/expansion to accommodate the recommended SDF, spillway improvement or replacement, and dam modifications to overcome any structural deficiencies of the embankment as necessary. Conduct periodic inspections of the project at 5-year intervals by engineers experienced in dam design and construction. Develop and implement a periodic operation and maintenance plan for the dam and appurtenant structures. Prior to performing engineering studies and remedial construction, the users are urged to coordinate the work with the Montana DNRC to insure compliance with all pertinent laws and regulations.

Mike Keene

Michael D. Keene
Professional Engineer

MICHAEL D.
KEENE
4197 E

PERTINENT DATA SUMMARY

1. General

Federal ID Number	MT 00017
Landowner and Project Administrator	Montana Department of Natural Resources and Conservation
Operator	Nevada Creek Water Users' Association
Date Constructed	1938
Purpose	Sedimentation, Recreation, Irrigation and Flood Control
Location	Sections 11 & 14, T12N, R10W, MPM
County, State	Powell County, Montana
Watershed	Nevada Creek (tributary to the Blackfoot River)
Hazard Potential	Category 1 (High)
Size Classification	Intermediate

2. Reservoir

Surface Area at Spillway Crest Elevation 4616.0 feet NGVD	368 acres
Storage to Low-Level Outlet Elevation 4551.5 feet NGVD	12 acre-feet
Original Usable Storage to Spillway Crest Elevation 4616.0 feet NGVD	12,640 acre-feet
Existing Usable Storage to Spillway Crest Elevation 4616.0 feet NGVD (reduced by siltation)	12,414 acre-feet
Surcharge Storage	4,660 acre-feet
Storage to First Overtopping Dam Crest Elevation 4627.5 feet NGVD	17,300 acre-feet
Storage to Design Dam Crest Control Elevation 4628.0 feet NGVD	17,530 acre-feet

PERTINENT DATA SUMMARY
(Continued)

Drainage Area	145 square miles
Water Surface at Time of Inspection	Approximately 4587.0 feet NGVD

3. Spillway

Crest Elevation	4616.0 feet NGVD
Type	Ogee crest with concrete chute
Crest Width	100 feet
Chute Width	30 feet
Spillway Capacity with Brush in Approach Channel, Existing Condition	
(to First Overtopping Dam Crest Elevation)	14,400 cubic feet per second
(to Dam Crest Control Elevation)	15,200 cubic feet per second
Spillway Capacity without Brush in Approach Channel	
(to First Overtopping Dam Crest Elevation)	15,900 cubic feet per second
(to Dam Crest Control Elevation)	16,800 cubic feet per second

4. Outlet Works

Control	By manual operation
Length	Total conduit and control chamber length = 392 feet
Conduit	209 feet from inlet to control chamber; 60-inch diameter reinforced concrete pipe with a 3/8-inch steel pipe liner
	26 feet downstream from the downstream transition section of control chamber; 60-inch diameter reinforced concrete

	pipe with a 3/8-inch steel pipe liner (same as above)
	137 feet of 60-inch diameter reinforced concrete pipe downstream of the lined section to the outlet
Control Chamber	20 feet long, located near center of dam, with two 6-foot transition sections upstream and downstream of the control gates; 2 control gates, a 54-inch slidegate valve upstream and a 54-inch pivot valve downstream.
Capacity	
With Reservoir Surface at Spillway Crest	860 cubic feet per second
With Reservoir Surface at First Overtopping Dam Crest Elevation	933 cubic feet per second

5. Dam

Type	Zoned earth fill
Structural Height	105 feet
Hydraulic Height	88 feet
First Overtopping Dam Crest Elevation	4627.5 feet NGVD
Design Crest Control Elevation	4628.0 feet NGVD
Crest Length (Excluding Spillway)	1083 feet
Crest Width	25 feet
Upstream Slope	1V on 3H; 1V on 2H
Downstream Slope	1V on 2H

CHAPTER 1
BACKGROUND

1.1 INTRODUCTION

1.1.1 Authority

This report summarizes the Phase I inspection and evaluation of Nevada Creek Dam. The project is administered by the Montana Department of Natural Resources and Conservation (DNRC), and operated and maintained by the Nevada Creek Water Users' Association. Reservoir lands are deeded to the State of Montana.

The National Dam Inspection Act, Public Law 92-367 dated August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to conduct safety inspections of non-federal dams throughout the United States. Pursuant to that authority, the Chief of Engineers issued "Recommended Guidelines for Safety Inspection of Dams" in Appendix D, Volume 1 of the U.S. Army Corps of Engineers' Report to the United States Congress on "National Program of Inspection of Dams" in May 1975.

The recommended guidelines were prepared with the help of engineers and scientists highly experienced in dam safety from many federal and state agencies, professional engineering organizations and private engineering consulting firms. Consequently, the evaluation criteria presented in the guidelines represent the comprehensive consensus of the engineering community.

Where necessary the guidelines recommend a two-phased study procedure for investigation and evaluating existing dam conditions so deficiencies and hazardous conditions can be readily identified and corrected. The Phase I study is:

(1) a limited investigation to assess the general safety condition of the dam

(2) based upon an evaluation of the available data and a visual inspection

(3) performed to determine if any needed emergency measures and/or if additional studies, investigations and analyses are necessary or warranted

(4) not intended to include extensive explorations, analysis or to provide detailed alternative correction recommendations.

The Phase II investigation includes all additional studies necessary to evaluate the safety of the dam. Included in Phase II, as required, should be additional visual inspections, measurements, foundation exploration and testing, material testing, hydraulic and hydrologic analyses and structural stability analyses.

The authority for the Corps of Engineers to participate in the inspection of non-federally owned dams is limited to Phase I investigations with the exception of situations of extreme emergency. In these cases the Corps may proceed with Phase II studies but only to the extent needed to answer serious questions relating to dam safety that cannot be answered otherwise. The two phases of investigations outlined above are intended only to evaluate project safety and do not encompass in scope the engineering required to perform design or corrective modification work. Recommendations contained in this report may be for either Phase II safety analyses or detailed design study for corrective work.

The responsibility for implementation of these Phase I recommendations rests with the State of Montana, Department of Natural Resources and Conservation. It should be noted that nothing contained in the National Dam Inspection Act, and no action or failure to act under this Act shall be construed (1) to create liability in the United States or its officers or employees for the recovery of damage caused by such action or failure to act or (2) to relieve an owner or operator of a dam of the legal duties, obligations, or liabilities incident to the ownership or operation of the dam.

The investigation process allows for report review by: the Montana DNRC (project administrator) and the Nevada Creek Water Users' Association (project operator). Review comments are considered before final publication of the Phase I Inspection Report. A written response from the DNRC is enclosed in Appendix F. The Water Users' Association did not wish to respond.

1.1.2 Purpose and Inspection

The findings and recommendations in this report were based on visual inspection of the project, minimal field survey measurements, and a review of available design and operation data. The purpose of the inspection was to make a general assessment as to the structural integrity and operational adequacy of the dam embankment and its appurtenant structures. Inspection procedures and criteria were those established by the Recommended Guidelines for Safety Inspection of Dams (Ref. 1).

The visual inspection of Nevada Creek Dam was made on September 25, 1979. HKM Associates personnel who attended the field inspection and contributed to this report were:

Dan Dyer, Geotechnical Engineer
Mike Keene, Hydraulics/Hydrology, Team Leader

Other HKM personnel contributing to the report but not attending the field inspection were:

Dale R. Cunnington, Structural Engineer
Dan Nebel, Geology
Kuo-Kung Shih, Hydraulics/Hydrology

Other personnel present during the inspection included:

Glen McDonald, Supervisor, Montana Dam Safety Section
Randy Hawkins, Ditch Rider, Nevada Creek Water Users' Association

1.2 DESCRIPTION OF PROJECT

1.2.1 General

Nevada Creek Dam is a compacted earth fill dam located in the SE¼ of Section 11 and NE¼ of Section 14, T12N, R10W, M.P.M., Powell County, Montana (Appendix A and Ref. 2). The dam and reservoir form a multi-purpose facility within the Columbia River Basin by containing the flows of Nevada Creek, a tributary of the Blackfoot River (Appendix B and Ref. 3). The nearest major community is Helmville, Montana, which is located approximately 8.5 miles northwest of the dam. The dam is constructed on property deeded to the State of Montana, with a total of 375 acres deeded for the reservoir pool right-of-way. The State of Montana and the Water Users' Association share in the operation and maintenance of the project (Ref. 4, 5, 6, 7).

Outflow facilities for the Nevada Creek Storage Project consist of an outlet works and spillway. The outlet works is a 392-foot long, 60-inch diameter conduit with gate controls located near the centerline of the dam axis. It has a maximum capacity of 933 cubic feet per second (cfs) with the reservoir pool at the existing low point on the dam crest. The spillway is located in the left abutment, and includes an earth-lined approach section, a concrete ogee crest, a concrete chute, and a stilling basin. Maximum spillway capacity under existing conditions to the existing low point on the dam crest is 14,400 cfs.

Nevada Creek Dam has a hydraulic height of 88 feet and impounds approximately 17,300 acre-feet (AF) with the reservoir surface at the first overtopping dam crest elevation 4627.5 feet NGVD. Based on a visual reconnaissance and engineering judgement, approximately 6 residential homes are located in the Nevada Creek flood plain between the dam and Helmville. In addition to the inhabitable structures, there are state, county and private roads and bridges; irrigation water delivery systems, and miscellaneous utilities that could be affected by a sudden breach of the dam. On the basis of this information and in accordance with the Recommended Guidelines (Ref. 1), the project is classified intermediate in size and the downstream hazard potential is high (Category 1).

Nevada Lake has capacity for sediment accumulation and floodwater detention, and provides irrigation water and recreational benefits. More specifically, the dead storage amounts to 12 AF and the total usable storage, to the spillway crest, currently amounts to 12,414 AF (Ref. 7, 8, 9). Flood surcharge storage between the spillway crest and the existing low point on the dam crest amounts to 4660 AF.

Nevada Creek Dam controls runoff from 145 square miles of drainage area, or approximately 6 percent of the total Blackfoot River watershed. The catchment area is comprised mainly of mountainous slopes and is classified as generally having a subdued topography. The higher slopes in the basin are forested with open grass-covered land adjacent to the valley floor. The valley floor is flat, open-meadow land except for dense willow thickets along the stream (Ref. 10, 11). The upper basin and the area bordering the reservoir are shown in Photos 7 and 8 of Appendix C. Elevations in the basin range from 7600 feet NGVD in the upper reaches to 4616 feet NGVD at the spillway crest.

1.2.2 Regional Geology

Nevada Creek drains mountainous areas formed on the northwestern flank of the Boulder batholith and associated structures. The drainage is situated between the Garnet Range and the southern terminus of the Swan Range. Precambrian sedimentary rocks form the core of the ranges with Tertiary volcanics having been extruded over the sediments along the flanks of the mountain masses. Surficial relationships indicate the presence of a large thrust fault with stratigraphic throw of probably more than 5000 feet through the Nevada Creek valley. The fault zone trends northwest and probably dips to the northeast. It crosses Nevada Creek about 1 to 2 miles below the damsite. The present relationship of the stream to the fault structure is probably due to superposition rather than structural influences, indicating that the fault has been inactive for a very long time, and that there is small probability of further movement along it (Ref. 10).

1.2.3 Seismicity

Nevada Creek Dam is in a moderately active to active seismic zone with the majority of the region's seismic events occurring in the Southwestern Montana-Yellowstone Park area. Since 1925, Montana has experienced five shocks that reached intensity VIII or greater (Modified Mercalli Scale). The closest epicenter occurred at Helena, Montana which is approximately 38 miles east of the damsite. Numerous other shocks of intensity IV or greater have been reported within a 100-mile radius of the site (as of January, 1974). The site is located in zone 3 of the Seismic Zone Map of Contiguous States, and it can be assumed that a major earthquake may occur within the life of the structure (Ref. 1, 14). Although the Zone Map is based on a known distribution of damaging earthquakes, it does not necessarily reflect accurate or adequate seismic design parameters for this site.

1.2.4 Site Geology

The U. S. Geological Survey explored the Nevada Creek Damsite in 1937. Location and logs of soil borings are presented in Exhibit D1 of Appendix D. Additional information is available in the project files, particularly in the Nevada Creek site geology report authored by Charles E. Erdman (USGS) and dated January, 1935 (Ref. 10).

Nevada Creek, at the damsite, flows through a narrow valley, the narrowing having been caused by rapid downward cutting of the stream through soft tertiary lake beds and encroachment of ancient landslide material onto the valley bottom. A thin basalt flow outcrops about 200 feet above the dam on both valley walls. Bedrock in the area is identified as being a hard broken quartzite underlain by slate and argillite and generally covered by slopewash and landslide detritus.

Based on information presented in Erdman, 1935 (Ref. 10) and Perry, 1939 (Ref. 11), the "material which underlies much of the dam, and the material against which the dam abuts at either end is a heterogeneous mass of broken rock, essentially irregular boulders of basaltic lava, and clay". This mass of lava boulders and clay is landslide material which has moved toward the stream channel down the valley slopes.

This founation material appears stable in character and has not disintegrated with wetting. Leakage was in evidence around the right abutment and beneath the dam during the early stages of project operation. A limited grouting program was completed in 1940, but apparently was not sufficient to stop the leakage (Ref. 10, 11, 12, 13).

1.2.5 Sedimentation

Lands above Nevada Lake consist primarily of hay meadows and brush along the valley floor, grazing land with sparse tree cover along the side hills, and dense forest cover in the mountain areas. As part of the original design, it was assumed that silting would occur but only in minor amounts. It was anticipated that the most abundant source of silt would be the Tertiary Lake Beds which outcrop a few miles above the site (Ref. 10).

A sediment study was conducted by the Montana Department of Natural Resources and Conservation in August 1973. From this study, it was estimated that the usable reservoir storage had been reduced from 12,640 AF to 12,414 AF, or by 226 AF (Ref. 8). Evidence of sedimentation in the upper reservoir basin can be observed on Photos 9 and 10 of Appendix C.

1.2.6 Design and Construction History

The Nevada Creek Storage Project was designed and constructed by the Montana State Water Conservation Board (SWCB). Construction of the project was financed by a Public Works Administration (PWA) loan and grant supplemented by SWCB funds (Ref. 15).

The first bid callings for project construction were rejected because they were not within the funding capacity. Items were deleted from the contract proposal to reduce costs. Bids were again received and on March 4, 1938 a contract was awarded to J. L. McLaughlin of Great Falls, Montana. The PWA approved the McLaughlin contract contingent on the SWCB guarantee that it would complete the deleted work items using its own funds. The contracted work began on April 1, 1938 and was completed December 7, 1938 (Ref. 15, 16).

Work items deleted from the contract included reservoir clearing, road construction on a side hill around the reservoir, and canal construction in the downstream valley. The additional work, except for the reservoir clearing, was accomplished with SWCB funds combined with funds and labor provided by the Works Progress Administration. Clearing of the reservoir site was not done (Ref. 15).

The project first operated on a limited basis in 1939 because construction work on the canal system had not been completed. Operation was also limited due to seepage observations and the necessity for grouting of the dam. Prior to the first reservoir filling, it was decided that considerable grouting should be performed to avoid excessive seepage in the dam.

The SWCB awarded a contract on June 15, 1940 to Caird Engineering Works of Helena, Montana. This contract was completed on August 14, 1940 (Ref. 15, 16, 17, 18). The SWCB authorized an additional expenditure for grouting on the dam on July 11, 1956 (Ref. 15), but available information is inconclusive as to whether the work was performed.

Recent maintenance at Nevada Creek Dam included the removal and repair of the 54-inch pivot valve (operating gate) in the outlet tunnel of the dam. The purpose of this work was to rehabilitate pitted and deteriorated areas on the operating gate and deflector. Under this same maintenance program, the 54-inch emergency gate was covered with a protective epoxy coating, the conduit metal lining was covered with a rust and wear resistant material, and the jet pump was repaired or replaced. J. A. Thompson & Son contracted with the Nevada Creek Water Users' Association to do the repair work. Work completion date was established as the spring of 1974 (Ref. 7, 8).

Other major repairs over the past few years have included patching the spillway and repairing the concrete weir below the outlet (Ref. 6).

CHAPTER 2
INSPECTION AND RECORDS EVALUATION

2.1 HYDRAULICS AND STRUCTURES

2.1.1 Spillway

The spillway for Nevada Lake is located in the left (south) abutment, with earthen dam embankment sections constructed on both sides (Sheet 1 of Exhibit D1). The spillway is a concrete lined chute having an earth-lined approach section and a concrete ogee crest. The 270-foot long approach channel is approximately trapezoidal in shape with a base width of 100 feet and a side slope of 1 vertical (V) to 2 horizontal (H), and is constructed on an adverse grade from the reservoir basin to the spillway crest. The ogee crest was set at elevation 4616.0 feet NGVD. Downstream of the ogee crest, the chute narrows from 100 feet to 30 feet within a distance of 215 feet (Sheet 3 of Exhibit D1). Within the 215 feet, the chute bedslope varies from a minimum of 6.9 percent to a maximum of 10 percent. For the next 175 feet, the chute slope is 12.75 percent. The final approach section to the stilling basin has a slope of 50 percent. Total chute length from the ogee crest to the stilling basin is approximately 500 feet. The spillway flows discharge into the stilling basin approximately 550 feet west of the downstream toe of the dam. The stilling basin is currently a rather large basin with scattered rock riprap and standing water. (Photos 34 and 36 of Appendix C, and Sheet 1 of Exhibit D1).

The design discharge capacity of the spillway was originally estimated to be about 9000 cfs with an 8-foot depth over the spillway crest (Ref. 8, 19). However, refined rating data for existing conditions indicates that only 8000 cfs can pass through the spillway at the 8-foot height. Maximum spillway capacity is estimated to be 14,400 cfs with the reservoir surface at the existing low point on the dam. The spillway rating data was developed using a weir head-discharge relationship with head-dependent discharge coefficients ranging from 3.25 to 4.09. Approach channel losses were considered insignificant when the channel is clear of brush, at which time the Mannings "n" value is about 0.028. For the existing conditions with a dense stand of brush and small trees, the "n" value is estimated to be 0.08 (Photo 32 of Appendix C). A friction loss formula was employed to estimate the friction losses from inlet to the ogee crest, and appropriate adjustments were made to the head-discharge relationship at the weir to establish the reservoir pool level associated with a certain discharge. Spillway rating data is provided in Exhibits E3 and E4 of Appendix E. Rating data is provided for both the "with brush" and "without brush" case.

The earthen approach section is currently covered with a relatively dense stand of brush and small trees (Photos 1, 3 and 32 of Appendix C). This condition applies specifically to an area approximately within 100 feet of the ogee crest. A log boom is not used, however, debris in the reservoir basin appears minimal. With the exception of the ogee crest and the stilling basin at the outlet, the concrete of the structure is in bad condition. The spillway floor has been a problem for a number of years, and even though some spot patching/repair has been performed, the concrete continues to weather, spall and heave (Plates 17 and 18 of Appendix C). The chute walls are in better condition than the floor but they too show extensive deterioration. The walls have numerous soft spots, extensive cracking and some spalling along the top edges (Ref. 8, 20). The stilling basin is in relatively good condition, with the exception of the following items: bank sloughing, particularly along the south bank where an almost vertical wall has formed; sparsity of rock riprap throughout the basin; slight undercutting at the concrete chute outlet; and the absence of positive relief in a downstream direction for stilling pool water, which has resulted in a stagnant condition.

It is not likely that the spillway chute walls would be overtopped during the maximum discharge condition. However, because of the poor structural condition of the walls and floor combined with the relatively severe hydraulic loading at peak discharge, it is likely that the structural integrity of the spillway would not be sustained. Fortunately the areas of the spillway which are particularly suspect in terms of structural integrity are comfortably removed from the dam embankment. Hence, even if failure occurred in these areas, the dam embankment should not be threatened.

2.1.2 Outlet Works

The outlet works for Nevada Creek Dam is located 250 feet from the right abutment contact and aligned approximately along the toe of the right valley wall (Sheet 1 and 2 of Exhibit D1). The outlet facility consists of a concrete intake structure with wrought iron trash rack, 60-inch diameter conduit, a dry control chamber and tower housing two 54-inch valves, a concrete outlet structure, a stilling basin, a control weir, and a return channel (Sheet 5 of Exhibit D1).

The inlet to the intake structure was set at elevation 4551.5 feet NGVD, which establishes a dead storage amount of approximately 12 AF. The outlet conduit is a reinforced concrete pipe with a 3/8-inch thick steel lining from the intake structure to approximately 26 feet downstream of the control chamber. From this point to the outlet, the conduit

is not steel lined, thus leaving a concrete exposure. Total conduit length from inlet to outlet is about 392 feet. The conduit was designed on constant gradient of about 0.13 percent from inlet to outlet (Sheet 5 of Exhibit D1). It was not possible to inspect the intake structure and the conduit upstream of the emergency operating valve due to the reservoir level, and hence, construction plan information could not be verified and physical conditions could not be established.

The control chamber, tower, and gatehouse are located at the center of the dam (Photos 3 and 4 of Appendix C). Access to the 90-inch diameter concrete tower is gained through the gatehouse situated on the upstream side of the dam crest. The control chamber is a "dry" setting, and houses a Chapman 54-inch gate valve and a Chapman 54-inch pivot valve. An air vent, 8 inches in diameter, is located immediately downstream of the pivot valve. The chamber is drained by a jet pump. Both valves are operated using Chapman PS-90-L #958 pedestal lifts.

The outlet structure consists of a concrete floor and concrete sidewalls. The sidewalls transition from vertical at the upstream end to a 1V on 1H slope at the downstream end. Floor elevation at the exit is 4540.0 feet NGVD. The physical configuration transitions to a stilling basin which includes a 20-foot long section with grouted paving and a raised channel invert to induce a hydraulic jump. The basin continues downstream for approximately 150 feet along a channel with flowline elevation of 4542.25 feet NGVD to a concrete control weir. The top of the 1/4-inch steel weir plate is 4246.0 feet NGVD. The weir structure also includes a low-level, or sluice, outlet and an irrigation service outlet. Flows drop from the weir crest to a channel invert elevation of 4540.0 feet NGVD and proceed for about 85 feet before joining the natural Nevada Creek channel (Sheet 4 of Exhibit D1).

Outlet works capacity data was obtained from the Montana DNRC (Ref. 21), and is summarized in Exhibits E3 and E4 of Appendix E. The outlet works head-discharge relationship was determined using methodology suggested in Reference 22. In particular, a form of the orifice equation was employed with appropriate consideration of trashrack, entrance, conduit friction, gate, and exit losses. Manning's "n" value varied from a minimum of 0.008 to 0.012.

The control valves and deflector have undergone repair several times in the past and again are in need of minor rehabilitation. The emergency valve shows a slight amount of leakage in the fully closed position and the deflector plate for the operating valve shows evidence of pitting on both sides. The seat area of the operating valve appears to

be in relatively good condition (Photos 27, 28 and 29 of Appendix C). The air vent appears to be performing satisfactorily and approximately meets minimum size criteria (Ref. 32 and 33 and Photo 28 of Appendix C). There is some pitting in the 54-inch by 60-inch cast iron reducer and the steel pipe lining immediately downstream of the valves. There are a few small leaks in the concrete conduit sections midway between the valves and the outlet (Photo 31). It appears that the outlet conduit has maintained relatively good horizontal and vertical alignment (Photo 30).

The outlet structure of the outlet works appears to be in good condition. No concrete spalling was evident and there is minimal undercutting of the structure. Seepage was occurring from the weep holes, particularly along the right side.

The channel between the outlet and the weir shows signs of erosion due to high flow velocities and bank subsidence due to spring activity. Erosion has occurred and sections of the grouted riprap have been removed within the stilling basin (Photo 15). Sections of this same grouted riprap have subsided along the right bank. Bank disconfigurations were observed even beyond the grouted sections on the right side. The spring activity and seepage in the area are the primary reasons for the slope failures and subsidence (Photos 20-26).

The concrete on the control weir has badly deteriorated, particularly on the upstream face. The 1/4-inch weir plate also exhibits deterioration, but is not critical. The 18-inch slidegate, intended as a low-level outlet or for sluicing operations, is rusted in the open position (Photos 16, 17, 18). The downstream sidewalls of the weir structure and the return channel to Nevada Creek are in relatively good condition (Photo 19).

2.1.3 Freeboard

The existing project is overtopped with the spillway design flood recommended by the guidelines (Ref. 1), and therefore has no freeboard for that condition.

The design normal freeboard for Nevada Lake from the spillway crest (4616.0 feet NGVD) to the first overtopping dam crest elevation (4627.5 feet NGVD) is 11.5 feet. The vertical distance from the water surface to the first overtopping dam crest elevation at the time of the September 1979 field inspection was approximately 40.5 feet. The historic highwater level for Nevada Lake was experienced in June 1953 when an elevation of 4618.3 feet NGVD was measured (Ref. 9), which results in a vertical distance to the first overtopping dam crest elevation of approximately 9 feet. This measured

level was somewhat verified by an elevation estimate of a debris line across the upstream face of the dam during the September 1979 investigation. Field measurements reduced to a common datum indicate the highwater level to be about 4618.0 feet NGVD, which confirms published measurements. It appears the minimum design freeboard was established at 4 feet, because the spillway design capacity was determined with an 8-foot depth over the spillway crest (Ref. 8). The fetch length in a southeast to northwest direction is about 2 miles, hence the minimum vertical distance between the spillway crest and the first overtopping dam crest elevation should be about 5 feet (Ref. 22).

2.2 HYDROLOGY

2.2.1 Physiography and Climatology

The catchment area above Nevada Lake is basically rectangular in shape and occupies about 145 square miles. Nevada Creek, at the damsite, flows through a portion of the valley which has been narrowed near the stream channel. The main valley at the site maintains an average width of about 1500 feet. The watershed rises from the lake by gradual slopes and bench lands to the higher mountainous elevations of the Garnet Range on the south and the Continental Divide on the north and east. The watershed divide along the southeast border is subdued compared to its neighboring divides. This divide is approximately 12 miles long and separates the Nevada Creek Valley and the Avon Valley.

Bottom land soils are generally dark colored, clay loams. As the bench lands are reached, more sand and gravel become evident. The surface soils along many of the benches are classified as sandy clay loams. The valley soil is alluvium containing a high percentage of gravel, cobbles and stones. Drainage is considered moderate to good throughout the area due to the underlying sand and gravel beds (Ref. 16, 17, 24).

The climate of the region may be classified as modified continental, occasionally reflecting tendencies to be similar to higher elevation Pacific slope climates. This area exhibits marked seasonal variations in climate, with differences in distribution of monthly average precipitation between mountain and valley areas. Specific to the moisture-runoff characteristic of the Nevada Creek basin, runoff is derived chiefly from melting snow. In general, the valleys are relatively dry during the colder months and wet during the late spring and early summer. The wettest part of the year in the mountains is generally from midwinter to early spring. Because there are no climatological stations located in the immediate vicinity of Nevada Lake, consideration is

given to regional measurements. The average annual precipitation for a climatological station near Ovando (elevation 4100 feet NGVD), which is located about 20 miles northwest of Nevada Lake, is 16.5 inches. For comparison, the average annual precipitation at a point about 18 miles southwest from Nevada Lake (Drummond Aviation, elevation 3940 feet NGVD) is 12 inches. Average annual temperature for both Ovando and Drummond is approximately 40°F. Winters are typically cold, with January being the coldest month. The monthly average temperature for January in Ovando area is 15°F, and 17°F at Drummond. Summers, although fairly warm, seldom produce oppressive heat. Even the warmest summer days are followed by pleasantly cool nights. July is commonly the warmest month of the year for this region, with average monthly temperatures ranging from 60°F to 65°F. The average number of days between the last recording of 32°F in the spring and the first in the fall is about 40 at Ovando. Using Deer Lodge (located about 28 miles south-southeast of Nevada Lake) as a comparison, its frost-free period is better than double that for Ovando (Ref. 9, 10, 16, 24, 25, 26).

Streamflow measurements have been maintained on Nevada Creek by the U.S. Geological Survey since 1939. The gaging station is located above Nevada Lake near Finn, Montana (gage number 12335500), and has an upstream contributary area of 116 square miles. Because of the lake's geographic and topographic setting, and the regional climate, the peak flows typically occur in the spring and early summer months. The largest peak of record occurred on June 2, 1953 when approximately 1800 cfs passed the gaging station. The 1953 spring runoff and flood was severe enough to also register the highest pool level of record at Nevada Lake (4618.3 feet NGVD on June 3, 1953) (Ref. 9).

2.2.2 Estimated Probable Maximum Flood (PMF)

The probable maximum precipitation (PMP) and the estimated probable maximum flood were developed for the Nevada Creek drainage basin. The ratio of the reservoir area to the non-reservoir area is less than 1 percent, therefore the two areas are not separated for the purpose of this analysis. The PMF is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the study region. The PMF is derived from the PMP with consideration of snowmelt contributions.

Nevada Lake is located west of the Continental Divide, and hence, PMP values were calculated using procedures contained in the National Weather Service Hydrometeorological Report No. 43 (HMR 43) (Ref. 27) and the National Weather Service

1967 memo (Ref. 31). More specifically, thunderstorm PMP values for durations up to 6 hours were estimated using HMR 43. The memo allows for some reduction of the PMP that was not originally contained in HMR 43. The 1-hour and 6-hour precipitations are 2.7 inches and 4.8 inches, respectively. It was found that the highest thunderstorm PMP value occurs during the month of August. Subdivision of the calculated rainfall increments was required to provide compatibility with the duration of the unit hydrograph. A 30-minute unit hydrograph was selected for Nevada Lake using criteria presented in the SCS Hydrology Handbook (Ref. 28). The August PMP storm was plotted in the form of a depth-duration curve for convenience in selecting incremental rainfall values. The 30-minute August PMP values were ordered according to the reverse pattern of the unit hydrograph ordinates. It is assumed that the PMP storm can be preceeded by another relatively severe storm that could cause the saturated condition. There is still potential for minimum retention losses to the soil. Therefore a loss rate of 0.15 inches/hour was used based on minimum loss rates for soils such as those found in the basin and prior storm conditions.

The runoff condition, or PMF, resulting from an August PMP thunderstorm was estimated using the PMP values and the unit hydrograph approach. A 30-minute unit hydrograph was developed for the Nevada Creek basin using the U.S. Army Corps of Engineers' computer program HEC-1 and the SCS method (Ref. 29). The resultant August PMF has a total runoff volume of 30,920 AF.

2.2.3 Flood Routing

The PMF resulting from the August rainfall/runoff event was routed through Nevada Lake using the computer program HEC-1 (Ref. 29). Runoff for the storm creating the antecedent ground conditions was not specifically considered; however, it appears reasonable to assume the initial reservoir level prior to routing the PMF is at the spillway crest (elevation 4616.0 feet NGVD). The support rationale for this assumption is that Nevada Lake characteristically has a small reservoir pool and is not capable of totally absorbing rather severe flood events.

Reservoir area-capacity-elevation data was obtained from the construction drawings and from DNRC files (Ref. 30). This data was slightly modified to accommodate sedimentation estimates, and incorporates the implementation of a different method to calculate capacity from measured areas (the conic method was used as compared to the previous method of average end-area). The modified data is provided in Exhibits E1 and E2 of Appendix E. Discharge rating data for Nevada Creek Dam hydraulic

structures is shown in Exhibits E3 and E4. Both the outlet works (valves in the fully open position) and spillway were utilized in the routing studies to pass flood flows.

For the purposes of the flood routing and according to Phase I investigation criteria, the dam crest elevation is the minimum elevation to which the reservoir must rise before overtopping the dam. This criteria assumes overtopping and failure of embankment-type dams to be coincidental. Based upon the dam crest profile survey dated October 5, 1979 (Exhibit D2 of Appendix D), the existing first overtopping dam crest elevation is 4627.5 feet NGVD. The elevation of the ground adjacent to the right wall of the spillway is 4626.6 feet NGVD, but this area is protected from overtopping by the concrete spillway headwall.

Flood routing showed that the dam would first overtop during the PMF when approximately 24 percent of the total PMF volume enters the reservoir. Routings were made of lesser hypothetical floods than the PMF to determine the magnitude of floods the project can safely contain. The hypothetical hydrographs are obtained by applying the same percentages to all PMF ordinates. A flood having ordinates corresponding to 34 percent of the PMF ordinates is just controlled by the project. Larger floods would overtop the dam. It should be realized, however, that the Nevada Creek Storage Project may have the storage and outflow capacity to accommodate flood events as severe as 34 percent of the PMF ordinates, but the project may not be capable of safely handling these floods. In particular, the spillway chute may not be capable of maintaining structural integrity under this type of loading. Anticipated damage to the spillway chute should be limited to that section sufficiently downstream that the dam embankment will not be threatened.

2.3 GEOTECHNICAL EVALUATION

The geotechnical evaluation of Nevada Creek Dam included a field investigation and a search and review of project design data. The field inspection consisted of photo documentation, a dam crest profile survey, slope stability observations of the dam embankment and the outlet works stilling basin, and measurements of the slope angles. Inspection photos are included in Appendix C, and the crest profile survey is shown in Exhibit D2 of Appendix D. Results of the slope angle measurements are provided in Exhibit D3.

2.3.1 Dam Embankment

Nevada Creek Dam is a compacted, zoned, earth fill structure which was completed in 1938. The dam has a structural height above the foundation surface of 105 feet and a crest

length and width of 1183 (including 100-foot wide spillway) and 25 feet, respectively (Photos 1-4 of Appendix C and Sheets 1 and 2 of Exhibit D1, Appendix D). The upstream face has a slope of 1 vertical (V) on 2 horizontal (H) down to elevation 4616.0 feet NGVD, which is the crest elevation of the spillway, then, according to the construction plans, flattens to a slope of 1V on 3H. Much of this flatter part of the upstream face was inundated at the time of this inspection. The upstream face is covered with loose, rock riprap and some scattered debris (Photos 3 and 38 of Appendix C).

The downstream face has a 1V on 2H slope. A short shale berm is located at the toe. There is a moderate to sparse cover of natural grasses, weeds and bushes on the downstream slope (Photos 5 and 6 of Appendix C).

The plans provided in Exhibit D1 of Appendix D (Sheet 2) indicate the embankment cross section consists of five parts identified as impervious, semi-impervious, pervious, riprap and riprap bedding, and crest road section.

The impervious section is a sloping zone located on the upstream side of the embankment. According to the construction drawings, a cutoff trench, having an approximate 30-foot bottom width, extending to bedrock or to a firm impervious strata, is located at the base of the impervious section. Available information indicates that the cutoff trench extends to or into bedrock under the main section of the dam but not in the abutment areas. The drawings indicate the portion of the cutoff trench in the bedrock is 5 feet deep and 10 feet wide. Test results indicate permeability rates of about 0.0038 feet per year in the embankment and cutoff trench material. According to the drawings, the impervious material was obtained from the borrow area immediately north of the dam (Photos 11 and 12 of Appendix C).

Based on the construction drawings, the semi-pervious section, which comprises the central section of the dam, is a shovel mixed material from borrow pits, foundation and spillway excavations.

The pervious section forms the downstream exterior face of the embankment. This material is sand and gravel with some boulders (Sheet 2 of Exhibit D1).

Detailed physical descriptions of the embankment zones were not available at the time of this investigation. Neither are records of quality control tests during construction available, however, the construction drawings indicate the materials in the pervious and semi-pervious sections were to be spread, sprinkled and rolled in 6-inch lifts (Sheet 2 of

Exhibit D1). The contractor had the option to spread and roll or place by sluicing, the material in the pervious section.

2.3.2 Foundation Conditions and Seepage Control

Soil boring logs for the dam foundation, abutment and borrow areas (Exhibit D1, Sheet 2) were reviewed. Soil tests were performed during the design period on selected samples taken from the test holes. These tests included: moisture-density relationships, specific gravity, grain-size distribution, consolidation, permeability, and porosity. Soil strength test results were not available.

Based on the construction drawings and the available information, it appears that the existing embankment is resting primarily on alluvial deposits of sand, gravel and clay. The core trench described earlier was designed to extend through the alluvial materials into the foundation bedrock or to "firm impervious material". Bedrock is a hard laminated red shale.

The abutments are heterogeneous masses of broken rock and clay. The drill logs indicate this material is about 70 and possibly more than 80 feet deep on the north and south abutments, respectively. This material is relatively permeable and appears stable.

Settlement

The construction plans indicate that the dam crest control was established at elevation 4628.0 feet NGVD. A road was constructed on the crest which required placement of a 12-inch wearing course. Thus, the final crest elevation was designed to be 4629.0 feet NGVD.

An embankment profile was surveyed on October 5, 1979 (Exhibit D2 of Appendix D). The survey shows that the maximum settlement along the crest is about 1.5 feet with an average of about 9 inches along the main section of the embankment. The point of maximum settlement is at the control tower (Photo 4 of Appendix C). This is likely the result of lower densities in the backfill around the tower and/or internal erosion of the embankment materials along the outlet conduit. The internal erosion may be related to the spring activity discussed in the next section, Seepage Control.

Settlement along the outlet conduit appears to have been minimal downstream of the control tower as the conduit alignment is very good (Photo 30 of Appendix C). Conduit alignment upstream of the control tower was not observed because it necessitates an almost complete reservoir drawdown for inspection.

Because of the age of Nevada Creek Dam, it is expected that additional settlement in the foundation and embankment materials will be minor, however, because of the localized settlement around the gate tower, and the possibility of related piping adjacent to the conduit, a few settlement momuments should be installed in this area to monitor any additional settlement and slope movements.

Settlement under the spillway has apparently been in the neighborhood of a couple of inches. While this settlement is not large, it may be a significant factor in reducing the service life of the spillway.

Seepage Control

Numerous springs surface at various places along the creek bottom immediately below the dam, and for several hundred feet below the embankment. Three areas were identified as having significant spring activity.

1) The outlet works stilling basin, particularly along the right (north) valley wall and channel bank immediately below the embankment,

2) Approximately in the middle of the valley and downstream of the dam near the underseepage collection pipe exit and drainage ditch, and

3) In the vicinity of the left abutment near the discharge of the spillway drain pipe.

These springs were first identified during the initial reservoir filling. Estimates made in 1939 of flows from these three areas totaled about 3.3 cfs. These estimates were made before the abutments were grouted in later years. Although precise measurements were not made during the September 1979 investigation, it appears these flows are still at least 3.0 cfs and possibly more. The grouting program appears to have been relatively ineffective.

Based on the information available, it appears that water from the reservoir finds its way downward into the sand, gravel and broken rock beds and may be traveling under and/or around the ends of the cut-off trench through the abutments. Visual inspection indicated that the resulting springs did not appear to be transporting embankment and abutment material at the time of this investigation. However, a piping-related problem has apparently developed in an isolated area on the right bank of the outlet works stilling basin. Piping from behind the mortared riprap section has caused subsidence, and consequently, cracking and disintegration of the mortar (Photo 26). It is also remotely

possible that the springs in the area of the outlet stilling basin may be related to piping (internal erosion) along the outlet conduit. Internal erosion along the outlet conduit is a potential cause for the settlement along the conduit which was identified previously in this report.

The springs along the north bank of the stilling basin exit at elevations as much as 10 to 12 feet above the channel bottom (Photo 24). A slope instability condition resulted in a small slope failure on the right bank of the outlet works stilling basin about 25 to 50 feet downstream from the concrete and mortar outlet structure. The location of the slide is shown on Exhibit D3 of Appendix D and Photos 21 through 24 of Appendix C. This slope failure will receive additional consideration in a later report section. The downstream toe of the embankment was dry in all areas except at lower areas of the outlet works at the time of this investigation. Significant movement or disfiguration in the area of the downstream toe of the embankment is not evident.

There are no piezometers in the embankment or foundation to monitor the location of the phreatic water surface.

Seepage through the embankment and some seepage through the abutments is collected in a drain pipe located at the base of the pervious downstream embankment section. The collected water exits into a drainage ditch located about 150 feet downstream of the toe of the dam (Exhibit D3, and Photo 14 of Appendix C). It appears that this seepage control system is operating near capacity based on the apparent flat gradient of the exit drain pipe and the drainage ditch hydraulics.

The springs on the left (south) side near the left abutment exit between 300 and 1000 feet downstream of the dam. The identified springs were numerous and scattered. High-volume discharges such as observed at the outlet works stilling basin were not evident.

2.3.3 Stability

The slope angles measured during the field investigation are shown in Exhibit D3 of Appendix D. These angles were measured with an Abney level and should be considered approximate.

The dam embankment slopes appear to be stable and in good condition at this time. The permanent cut-slopes in the old borrow area north of the dam are steep and slightly eroded on the face. These borrow area cut-slopes appear too steep to support good vegetative growth (Photos 11 and 12 of Appendix C).

A small slope failure was observed on the right bank of the outlet works stilling basin as noted earlier. The scarp is about 1 to 2 feet high at the top of the slope. The toe of the slope has moved into the stilling basin causing some channel disfiguration. The slope failure appears to have been initiated by spring activity. The riprap in the channel floor appears to be underlain by fine-grained silt and clay.

Visual inspection of the embankment indicates no evidence of instability, however, sufficient information is not available to fully assess the stability of the embankment. No stability, seismic, or seepage analyses are available for review. There are no piezometers in the embankment to determine the location of the phreatic water surface.

The reservoir shorelines are considered to be basically in stable condition as no recent major slides or scarpments were observed. The ancient landslides in the reservoir area appear stable. The upland slopes are generally flat, with only the shoreline along the county road exhibiting a high degree of steepness (Photos 1, 2, and 7 - 10 of Appendix C).

2.3.4 Rock Riprap

Rock riprap has been placed on the upstream face of the dam, in the outlet works stilling basin and along the banks of the return channel, along the banks of the drainage ditch, and in the spillway stilling basin. This rock is classified as a quartzite sandstone, which is generally erosion resistant.

The rock was generally in excellent condition at most sites (Photos 3, 14, 15, 19, 20, 34, 36, 37, and 38 of Appendix C). However, there were some rock size and quantity deficiencies at some of the sites; in particular, along the right bank and on the bottom of the outlet works stilling basin (Photos 15 and 20 of Appendix C), and in the spillway stilling basin (Photos 34 and 36 of Appendix C).

2.4 PROJECT OPERATIONS AND MAINTENANCE

The Nevada Creek Storage Project is administered by the Montana DNRC through a contractual agreement with the Nevada Creek Water Users' Association (Ref. 7, 15). Initially, the State Water Conservation Board (SWCB) had the responsibility to construct, operate and maintain water conservation projects. In 1967, the SWCB was replaced by the Montana Water Resources Board (MWRB). The new agency shifted its emphasis from construction activities to providing engineering services for local groups and water user associations, to assisting in the maintenance of previously constructed projects, and

to conducting annual maintenance inspections. Responsibility for project operation and maintenance gradually shifted from the State agencies to the local groups over a period of years. Upon reorganization of state government in 1971, the MWRB was replaced by the Department of Natural Resources and Conservation (DNRC). Accounting and engineering responsibilities were departmentalized, with the Engineering Bureau of the Water Resources Division, DNRC, delegated the responsibility to provide the services previously mentioned for the MWRB. At the present time, the Nevada Creek Water Users' Association performs essentially all operation and maintenance activities and the state agency serves only in a supervisory capacity (Ref. 7).

Representatives of DNRC maintain frequent contact with the water user group by attending pertinent meetings, and by scheduling regular inspections of the project. They also advise the group of maintenance, repair, operation and budgetary needs. The DNRC provides assistance and direction in determining the best repair alternatives and financing sources (Ref. 7).

Releases from the reservoir are generally dictated by seasonal requirements. Seasons of particular interest are the spring, early-summer runoff period and the irrigation period. The project operator generally considers the poor condition of the spillway when regulating the reservoir prior to the flood season. Releases during the irrigation season are monitored and controlled according to water needs and rights. Reservoir pool carry-over from year to year also plays a part in project operation. No operations manual for the project has been prepared, and operating records are kept only on some of the components for the total storage and water delivery system.

Miscellaneous maintenance work has been accomplished in the past on the dam and related structures. However, this activity has been limited due to manpower and budgetary constraints.

2.4.1 Dam

Periodic maintenance of Nevada Creek Dam is performed "as needed". The grouting work to reduce underseepage is the most significant enhancement or repair performed since the dam was completed in 1938. Grouting was performed in 1940, approximately 18 months after dam completion. The SWCB again authorized an expenditure for grouting on the dam in July 1956 (Ref. 15), but available records could not confirm that the additional work was completed.

2.4.2 Reservoir

The U.S. Geological Survey maintains a gage on Nevada Creek upstream from the reservoir, which provides a fairly complete description of inflows. The reservoir water surface elevation is read at or near the end of every month by the DNRC. The DNRC also maintains a gaging station on the river below the dam and on some of the major canals. Deliveries are made by a water commissioner with the aid of measuring devices and the gate stem height on the outlet works operating valve (Ref. 7, 8).

Maintenance and repairs have not been required in the reservoir basin. A sediment survey was performed by the DNRC in 1973, and the results provided an approximate quantification of sediment accumulations in the upper reservoir basin (Ref. 7, 8).

2.4.3 Related Structures

The spillway is an uncontrolled facility. All other related structures, such as the outlet works and irrigation turnouts, are controlled by the local water commissioner.

Repairs have been made on the 54-inch emergency and operating valves, the deflector plate on the operating valve, and the jet pump. Repairs have also been reported for the spillway and the concrete measuring weir below the outlet (Ref. 6, 7, 8). Of greatest concern in recent time has been the condition of the spillway, but little has been done because of anticipated changes in spillway design flood criteria and budgetary constraints (Ref. 8, 20).

2.4.4 Warning System

There is no formal warning system or plan of action in the event of dam distress.

CHAPTER 3
FINDINGS AND RECOMMENDATIONS

3.1 FINDINGS

Visual inspection of the dam, supplemented by analysis of the project in accordance with the guidelines (Ref. 1) and the contract performance standards, resulted in the following findings.

3.1.1 Size, Hazard Classification and Safety Evaluation

The dam, with a hydraulic height of 88 feet, impounds approximately 17,300 AF at the first overtopping dam crest elevation. A sudden breach of Nevada Creek Dam could endanger many lives and cause extensive damage to agricultural land, transportation facilities, and utilities. On the basis of this information, and in accordance with the guidelines (Ref. 1), Nevada Creek Dam is classified intermediate in size with a high downstream hazard potential rating (Category 1). The recommended spillway design flood (SDF) for this project is the full PMF. The project can only handle a flood event equal to about 34 percent of the PMF hydrograph ordinates. Because the project is incapable of controlling 1/2 the PMF without overtopping and causing the dam to fail, Nevada Creek Dam does not meet inspection guidelines and is considered to have a seriously inadequate spillway and the project is considered unsafe, non-emergency, until recommended actions (Section 3.2) are completed.

3.1.2 Spillway

The spillway system was designed to accommodate a relatively small flood event compared to current standards. The spillway is in very poor condition, particularly with respect to concrete deterioration. Brush has accumulated in the spillway approach channel and consequently affects the hydraulic conveyance. Slope and bank erosion has occurred in the stilling basin, along with material aggregation and degradation.

The PMF for the 145 square-mile drainage area is estimated to have a total runoff volume of 30,920 AF. Maximum combined outlet works and spillway capacity, assuming the reservoir pool is at the existing first overtopping dam crest elevation and the spillway facility can withstand the severe loading, is approximately 15,333 cfs. Floodwater storage capability consists of 4660 AF between the spillway crest and the first overtopping dam crest elevation. The combination of reservoir storage and spillway discharge capabilities is inadequate to prevent overtopping of the dam during the PMF.

3.1.3 Outlet Works

The outlet works facility appeared to be in satisfactory operating condition, however, the portion of the outlet works upstream from the gate tower could not be inspected. Minor repair is required on the gates, the deflector plate, the concrete section of the conduit, and the riprap in the stilling basin. More extensive repair is required along the right bank of the stilling basin where a portion of the grouted riprap has subsided due to spring activity and piping.

3.1.4 Project Related Structures

Several problems were observed at the control weir downstream of the pipe outlet. The 18-inch slidegate on the low-level outlet is rusted in the open position, and permanently inoperable due to metal deterioration on the gate and frame. The concrete on the weir structure is badly deteriorated particularly on the upstream face. The staff gage upstream of the weir structure should be replaced and recalibrated.

3.1.5 Dam

The embankment structure showed no sign of instability and appears in good condition, however, there is insufficient information to fully evaluate the stability. The spring activity immediately downstream of the embankment, the amount of seepage exiting from the collection system, and seepage exiting under pressure through the outlet conduit and outlet works return channel is of some concern. No seepage or stability analyses for the dam embankment and abutments are on file. Large and small debris was evident on the upstream face of the dam embankment.

3.1.6 Sedimentation

The only monitoring of sediment yields and deposition in the reservoir pool consisted of a field investigation performed by the Montana DNRC in 1973. They found that the reservoir usable contents should be reduced approximately 226 AF; that is, from 12,640 AF to 12,414 AF. There is little doubt that sedimentation is occurring, as evidenced in Photos 9 & 10 of Appendix C. The rate, the distribution, and the total extent of sedimentation remains unknown at this time.

3.1.7 Operation and Maintenance

The dam is visited regularly throughout most of the year by the ditch rider and/or water commissioner to regulate water releases. Also, the reservoir water surface is measured

once a month at the measuring devise on the right (north) shoreline near the dam. Regular project inspections are performed jointly by the DNRC and representatives of the local water users' association. Some repair work has been accomplished in the past, but not all project components have experienced maintenance when first required. The basic reason for the meager maintenance programs and repair efforts is the ever-present budgetary constraint.

3.2 RECOMMENDATIONS

(1) Immediately develop, implement, and periodically test an emergency warning plan for use in the event of dam distress.

(2) Accomplish short-term and minor improvements in the spillway; such as, brush removal in the approach channel, concrete patching where required to maintain structural integrity, rearrangement/replacement of rock riprap in strategic areas of the stilling basin, and providing positive drainage from the basin. See Appendix F for work done subsequent to the September 1979 inspection.

(3) Practice maximum reservoir management to prevent large discharges from flowing through the existing spillway until it is repaired or replaced.

(4) Inspect the outlet conduit from the intake structure to the valve chamber and repair as needed.

(5) Perform minor repair work on the 54-inch emergency and operating valves, the deflector plate on the pivot valve, and the leaky sections of the outlet works conduit.

(6) Repair the outlet works stilling basin, which should include: rebuilding the riprap, particularly where the mortared section has deteriorated; rehabilitating the mortared, rock riprap section along the right bank which has subsided; stabilizing the right bank by flattening the areas with steep slopes and/or by placing a thick granular section over the fine-grained soils; reshaping the right bank where a slope failure has occurred and replacing the riprap; and rearranging and/or replacing rock riprap immediately downstream of the concrete outlet structure to eliminate future scouring and undercutting.

(7) Install piezometers in the embankment, abutments and foundation to evaluate the seepage conditions. Establish a regular monitoring program for the under-

seepage, abutment seepage, and spring activity. Evaluate whether or not the springs at the toe and below the dam are causing piping of materials. Particular attention should be given to periods when the reservoir pool is relatively high.

(8) Establish reference points along the embankment and at exterior sites to monitor long-term settlements and slope movements.

(9) Periodically remove large debris from the upstream face of the dam.

(10) Repair the concrete on the upstream face of the control weir.

(11) Replace the 18-inch slidegate and frame on the weir structure.

(12) Replace and recalibrate the staff gage upstream of the control weir.

The above recommendations will not make the project safe but will reduce involved risks while the following recommendations with subsequent actions are being accomplished.

(13) Conduct engineering studies to better determine the downstream hazard and required spillway capacity and modify the project as studies indicate.

(14) Conduct periodic inspections by qualified engineers at least once every five years to determine whether there are any deficiencies in the condition of the project, to assess the adequacy and quality of maintenance, and to evaluate methods of operation. Include an inspection of the total length of conduit under the embankment in this program. In addition to the recommended periodic inspection, the dam should be inspected after any moderate size earthquake events occurring near the damsite.

(15) Develop and implement a periodic operation and maintenance plan for the dam and appurtenant structures.

(16) Conduct and place on file stability and seepage analyses of the dam embankment. These analyses must be performed by a qualified geotechnical engineer and be based on seismic and static loading conditions, insitu strength properties of the embankment, foundation and abutment materials and phreatic surface conditions. The materials strength properties must be established by a drilling sampling and laboratory testing program

and the phreatic surface obtained by monitoring piezometers. Modify the embankment as these studies indicate to provide adequate factors of safety for all loading conditions.

Prior to performing engineering studies and remedial construction, the users are urged to coordinate the work with the Montana DNRC to insure compliance with all pertinent laws and regulations.

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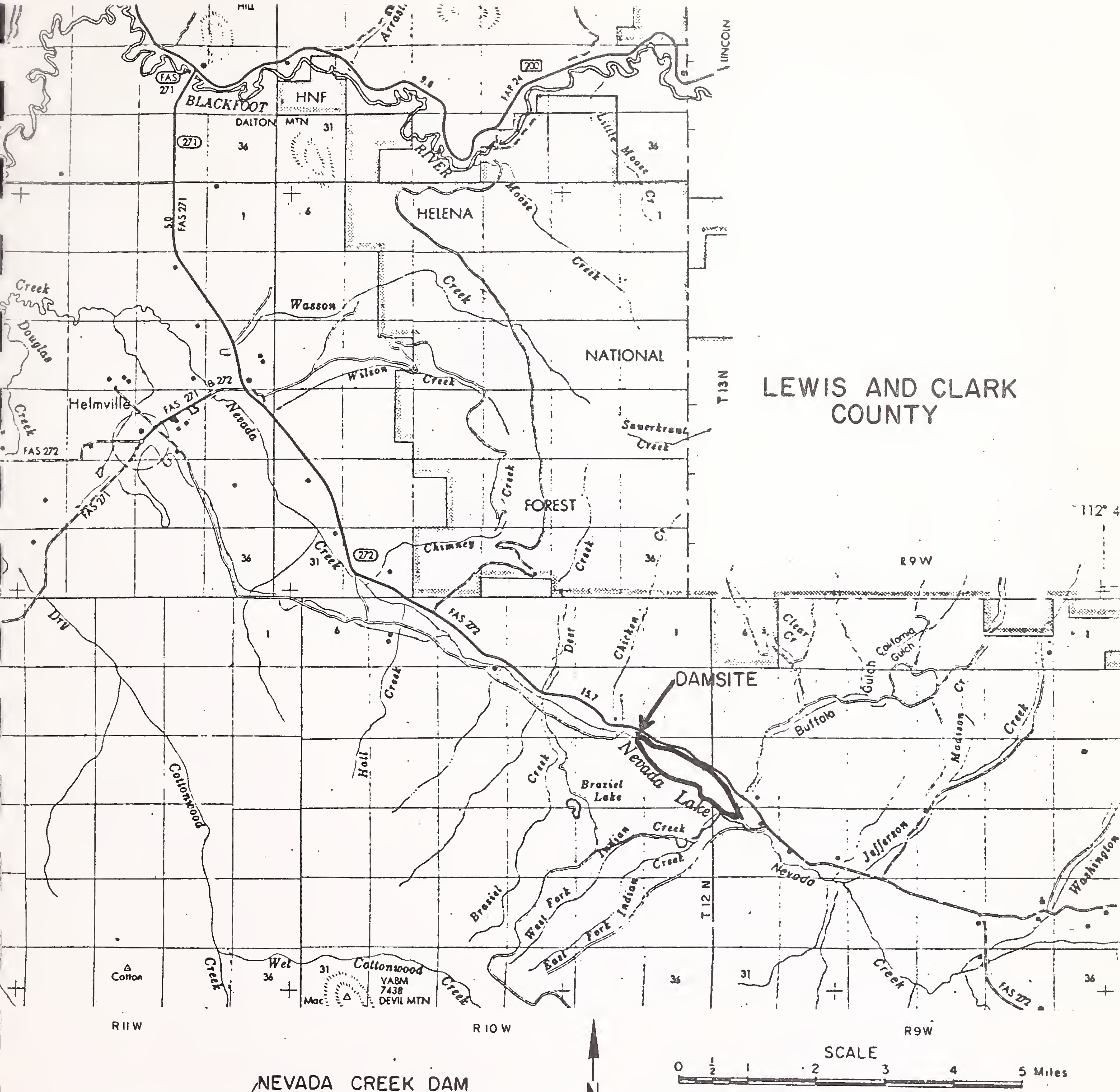
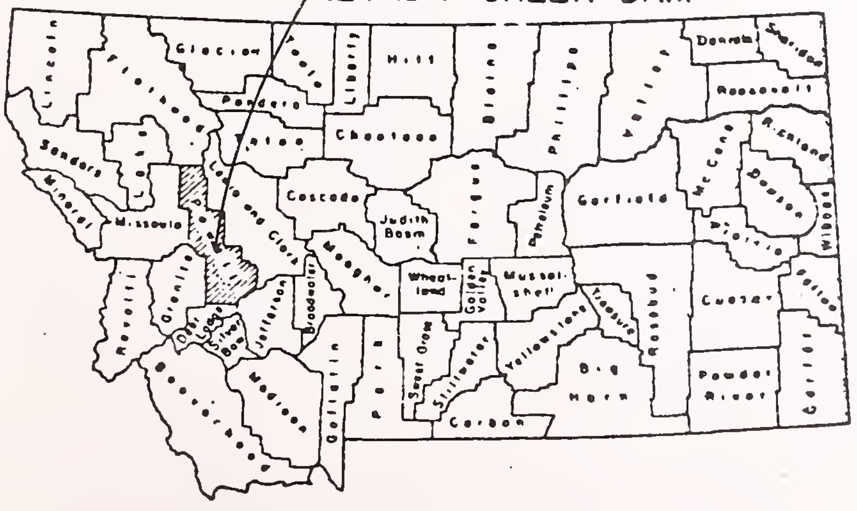
APPENDIX A

NEVADA CREEK DAM
VICINITY MAP



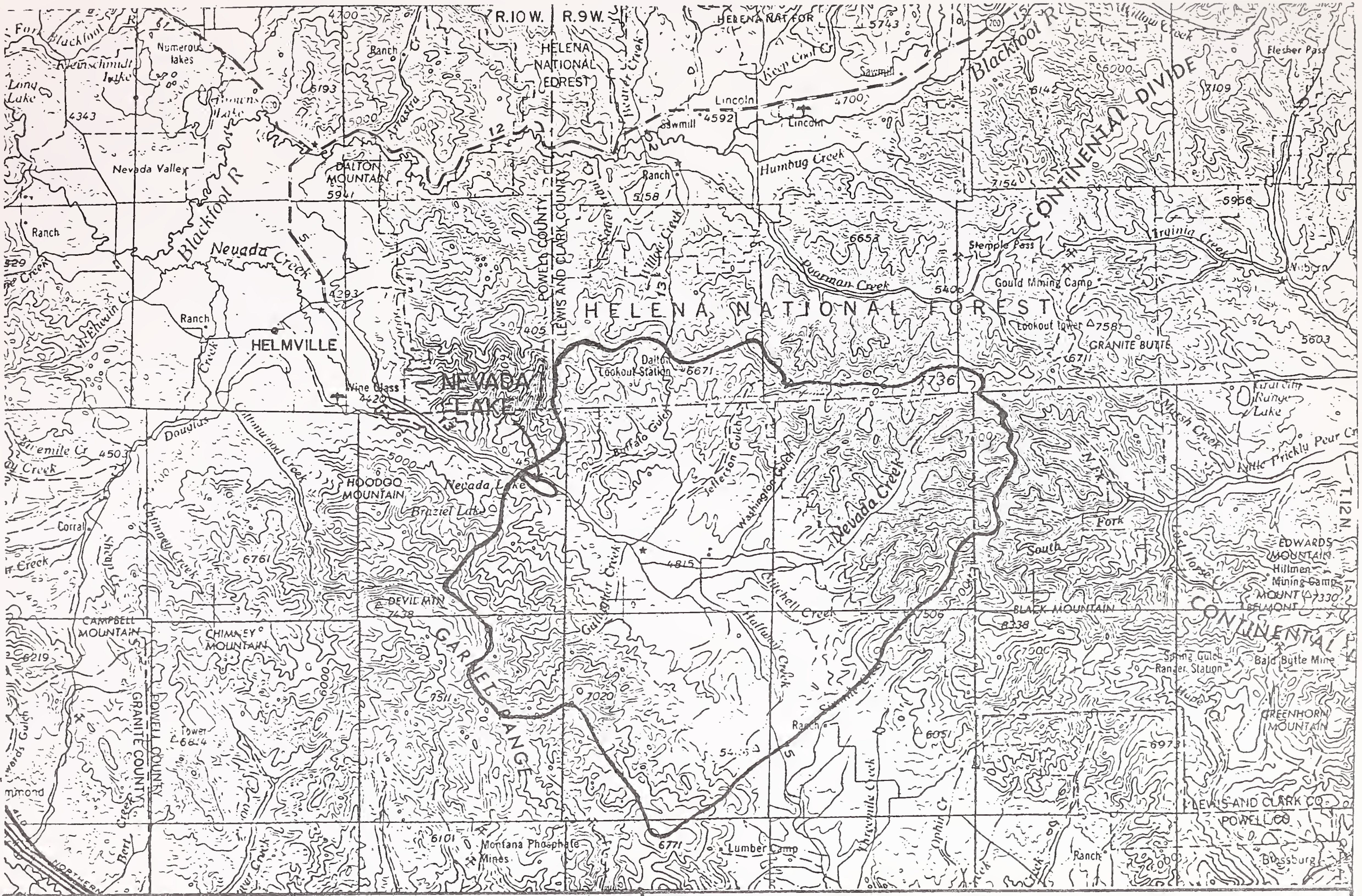


SOURCE: GENERAL HIGHWAY MAP, (1973)
POWELL COUNTY, MONTANA
MONTANA STATE HIGHWAY COMMISSION

APPENDIX A
VICINITY MAP
NEVADA CREEK DAM

APPENDIX B

NEVADA CREEK DAM
WATERSHED MAP



APPENDIX B
NEVADA LAKE WATERSHED MAP

(EXCERPT FROM BUTTE AMS MAP, U.S.G.S.)

APPENDIX C

NEVADA CREEK DAM INSPECTION PHOTOS

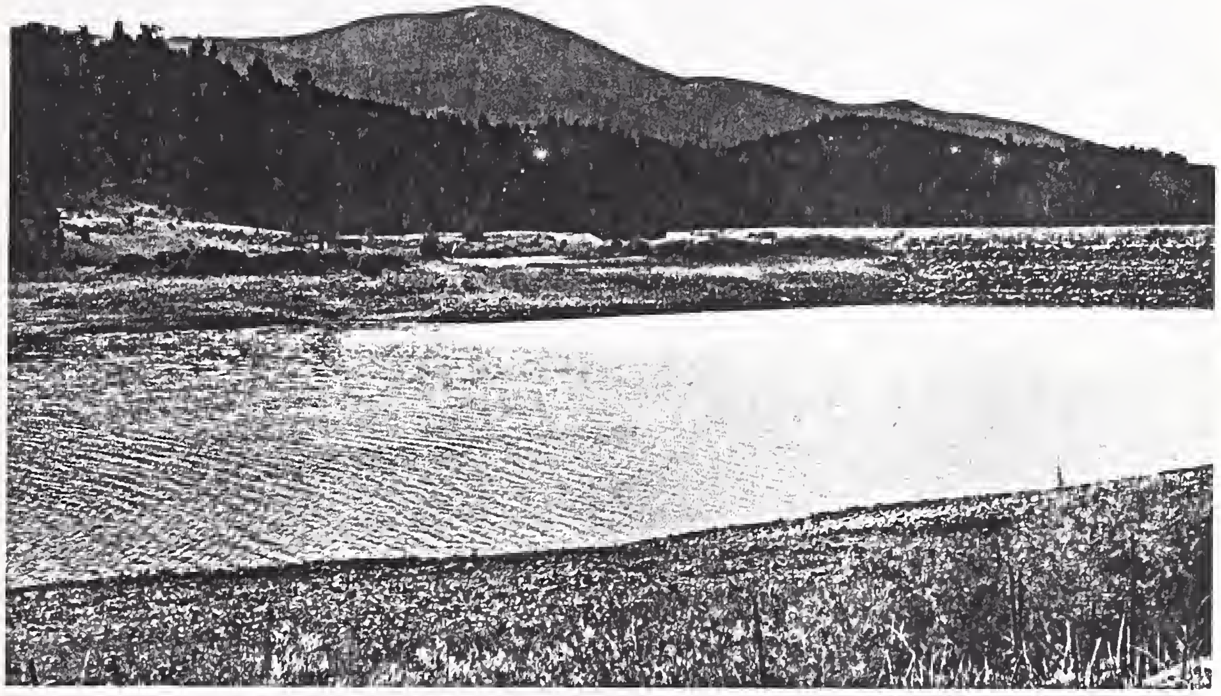


PHOTO NO. 1 - Upstream Face of Dam and Emergency Spillway Appoach Channel. First in a two-picture series. Normal highwater line on dam face is clearly visible. Note the brush in the emergency spillway approach channel.

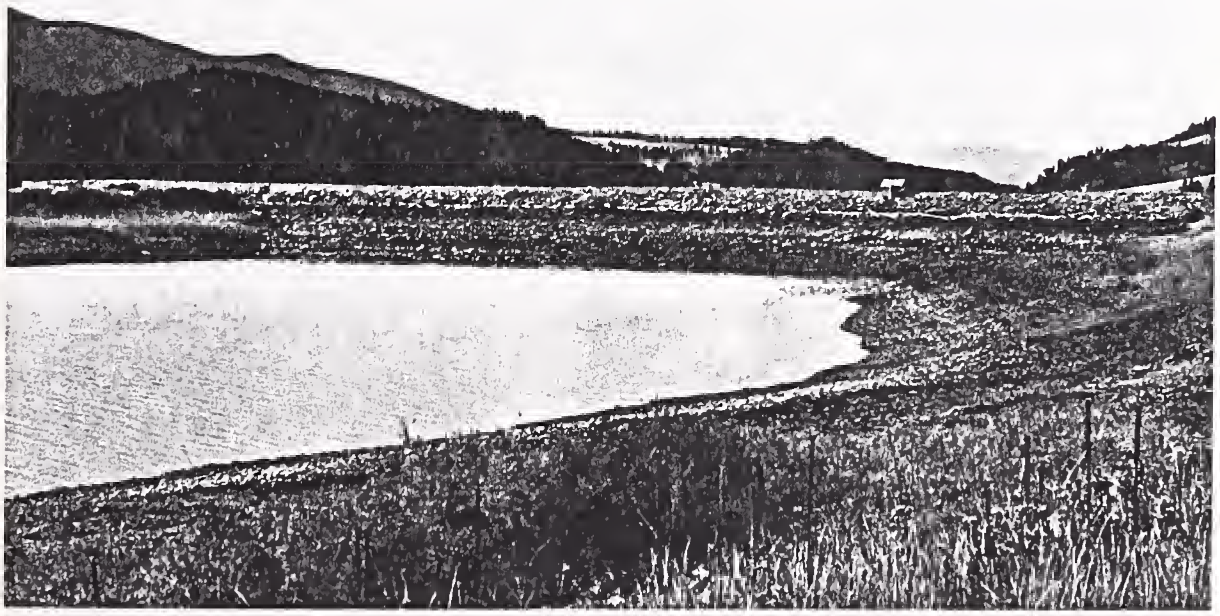


PHOTO NO. 2 - Upstream Face of Dam. Second in a two-picture series (See Photo No. 1 for first frame).

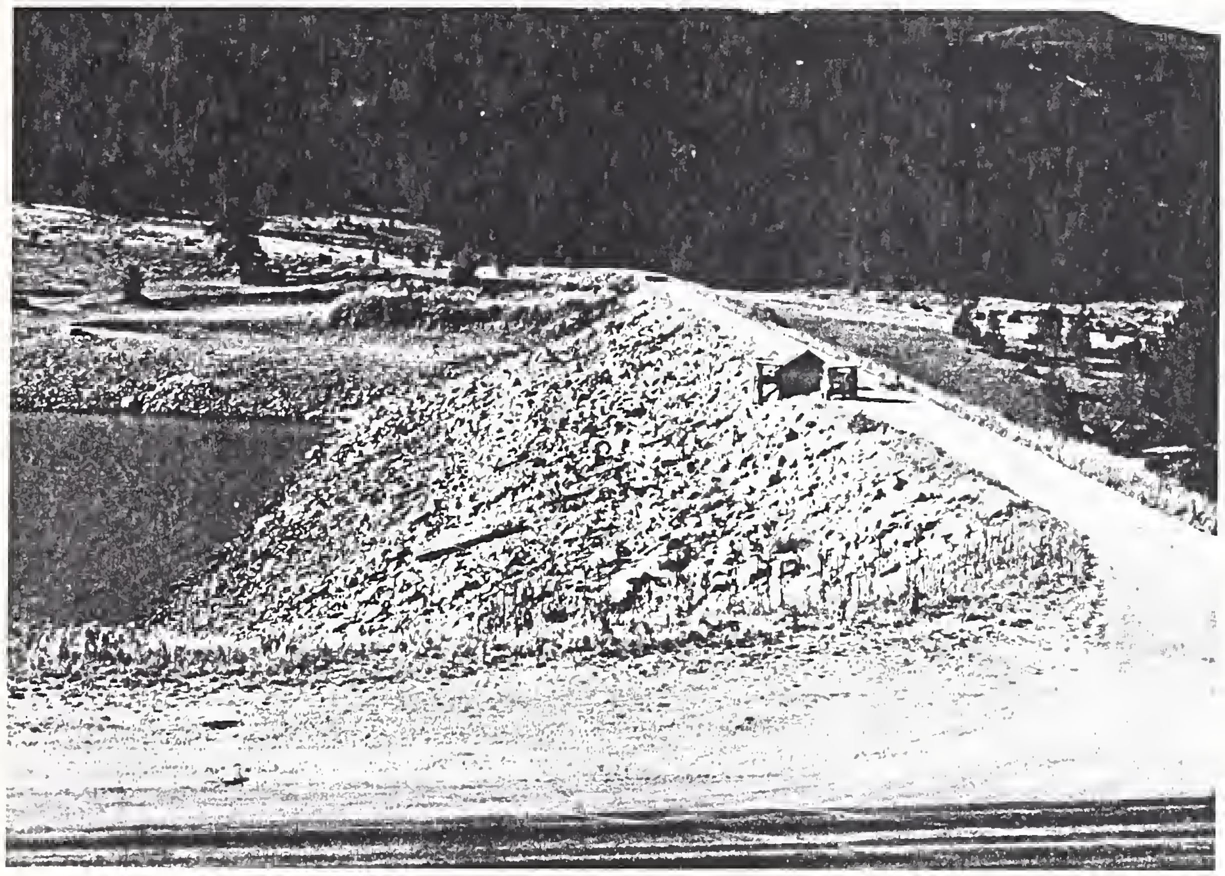


PHOTO NO. 3 - Upstream Face of Dam.
Notice debris accumulation and a high-water mark. Looking southwesterly toward the left abutment. Emergency spillway approach channel in left-center of photo.

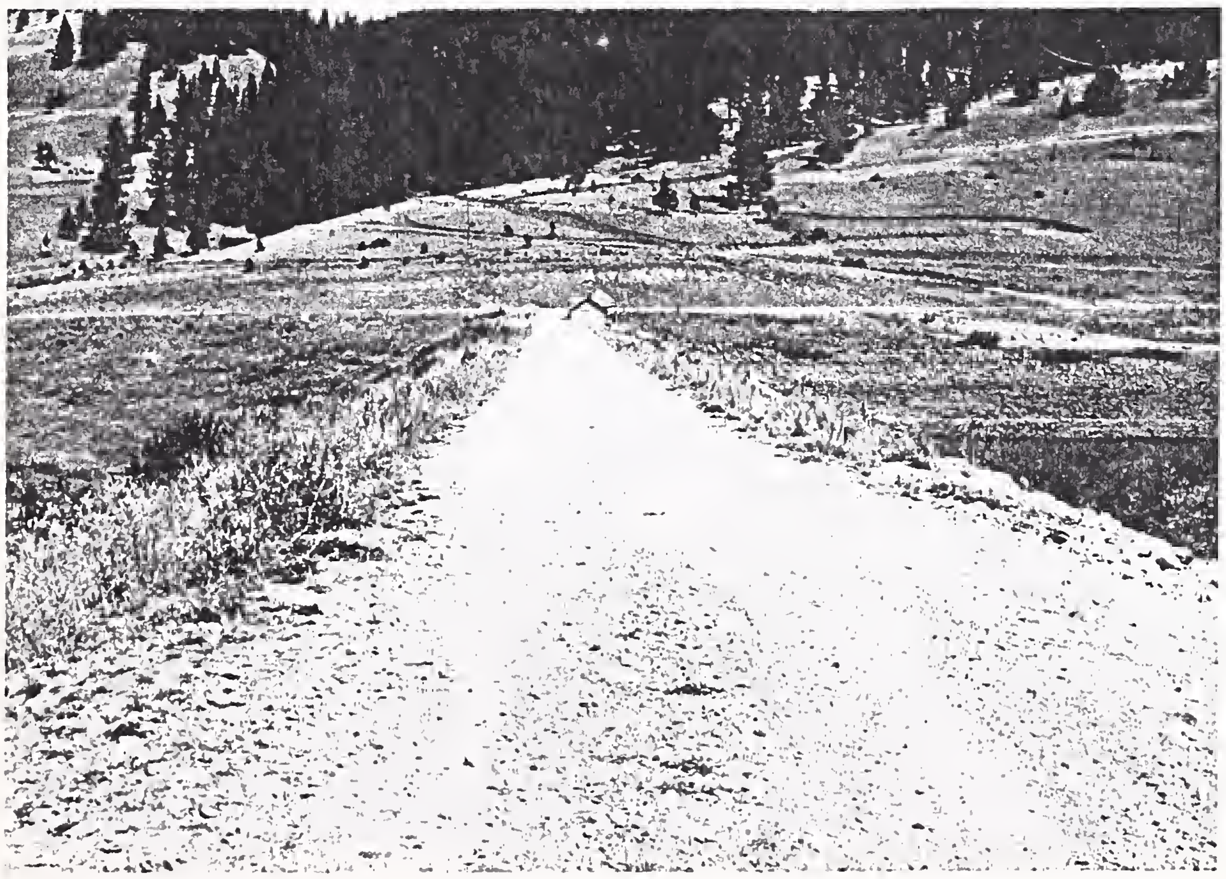


PHOTO NO. 4 - Dam Crest.
The crest has a gravel surface course with some depressions. Standing at the left abutment looking to the right side.

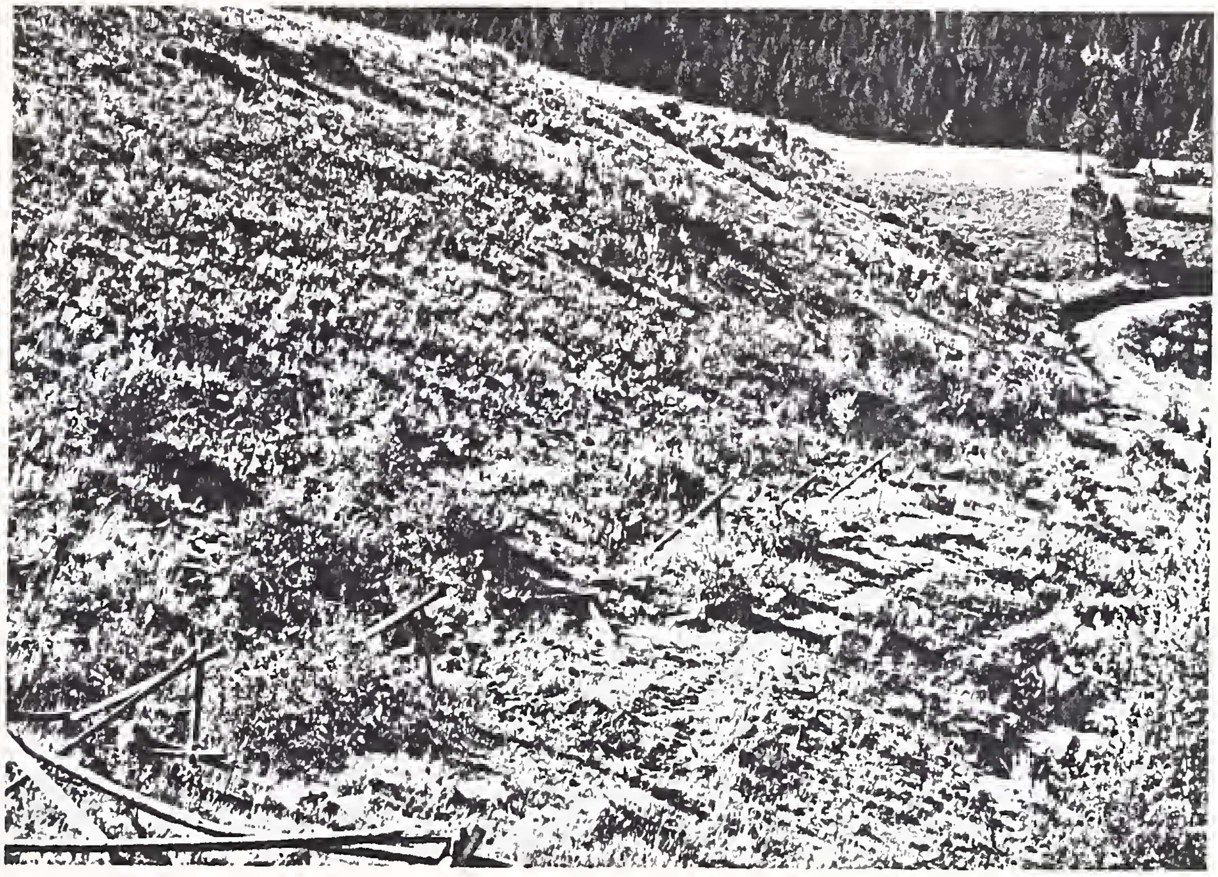


PHOTO NO. 5 - Downstream Face of Dam.
Looking toward the left abutment. Notice type and extent of vegetative cover.

PHOTO NO. 6 - Downstream Face of Dam.
Looking toward the right abutment. The face has a moderate to sparse cover of natural grass, weeds, and young trees.

PHOTO NO. 7 - Reservoir Basin.
Standing in the emergency spillway approach channel looking upstream into the reservoir basin.

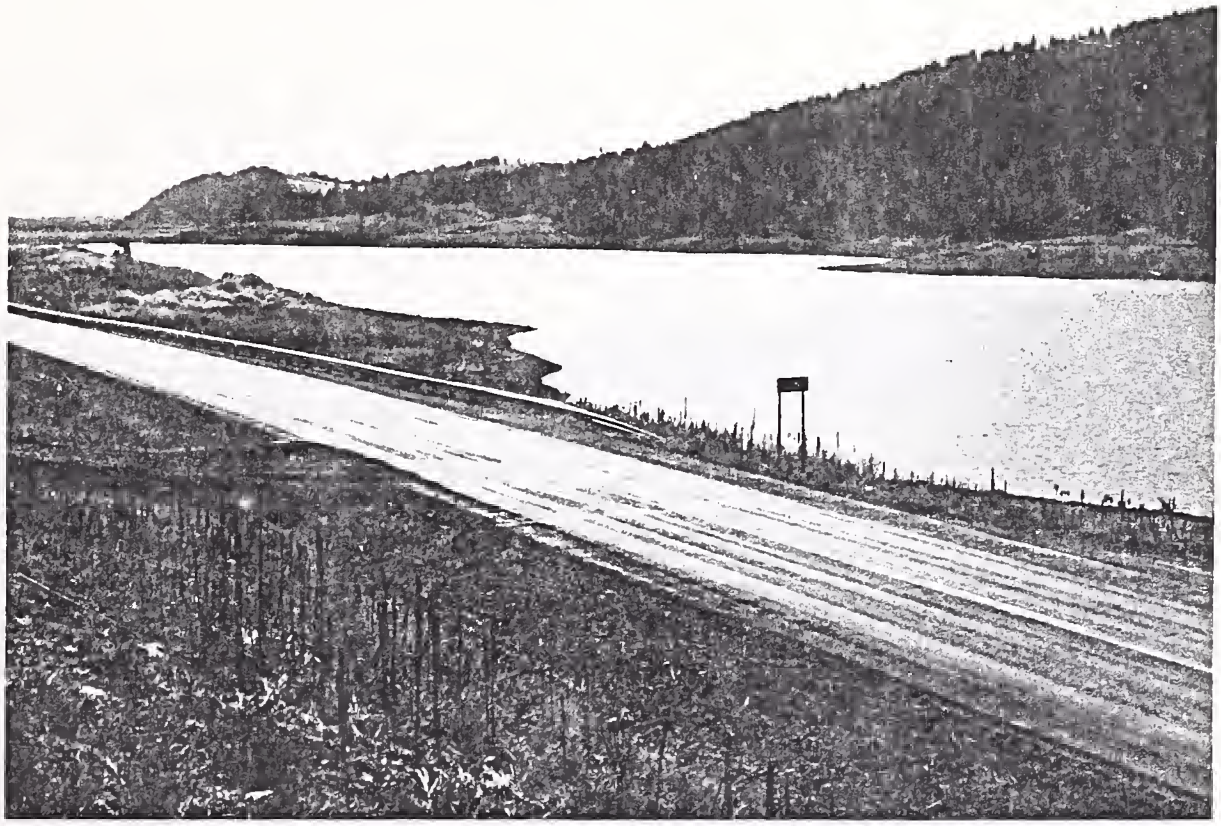


PHOTO NO. 8 - Reservoir Area and Shoreline.
Standing on the right side of the reservoir looking upstream. The drainage area is characterized by a combination of forest, farm, and range lands. Note the shoreline slopes.

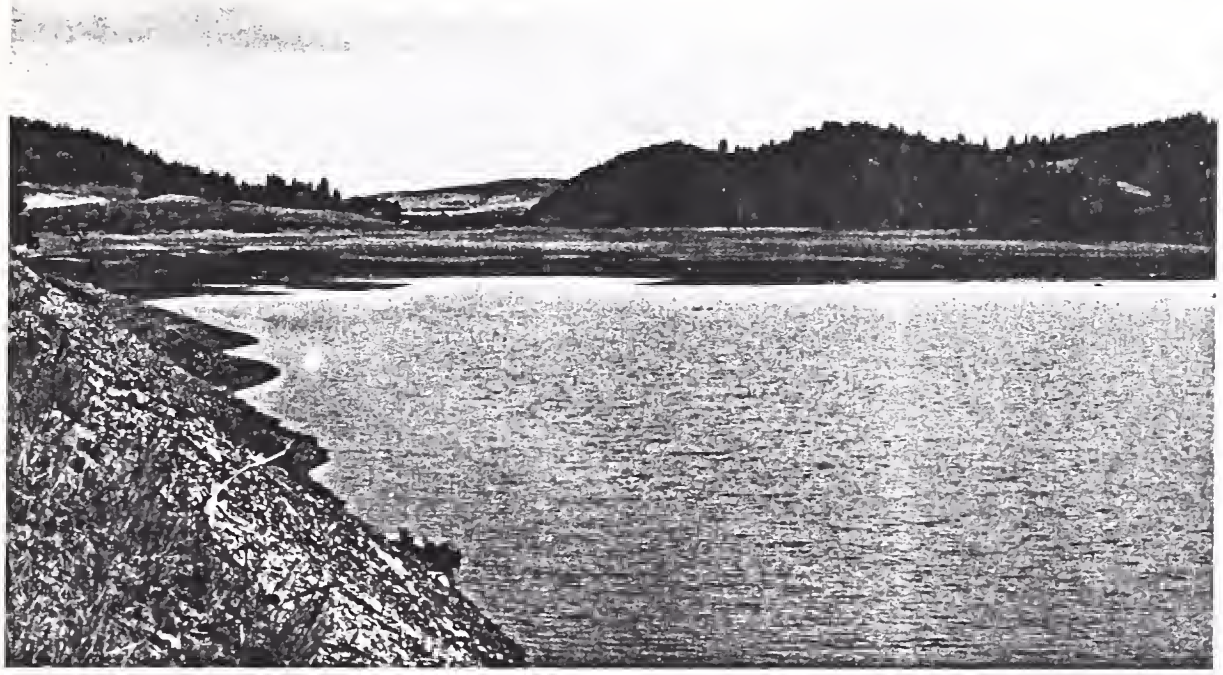


PHOTO NO. 9 -- Upper Reservoir Basin.
Notice the siltation. Standing on the county road looking south.

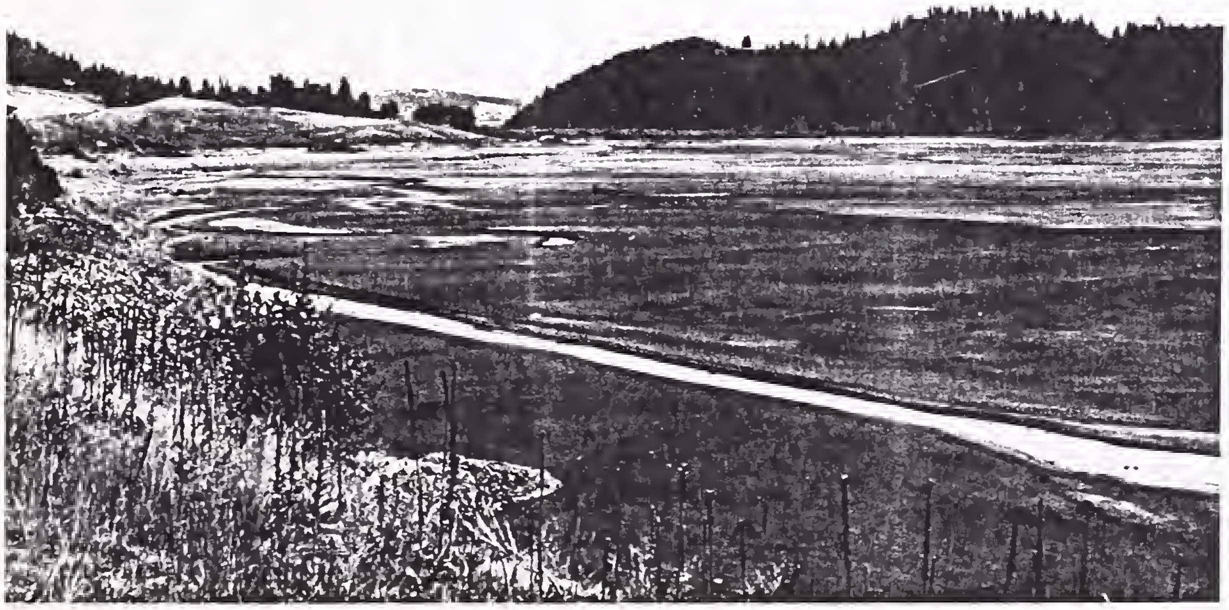


PHOTO NO. 10 - Siltation in the Upper Reservoir Basin.
Standing on the county road looking south.

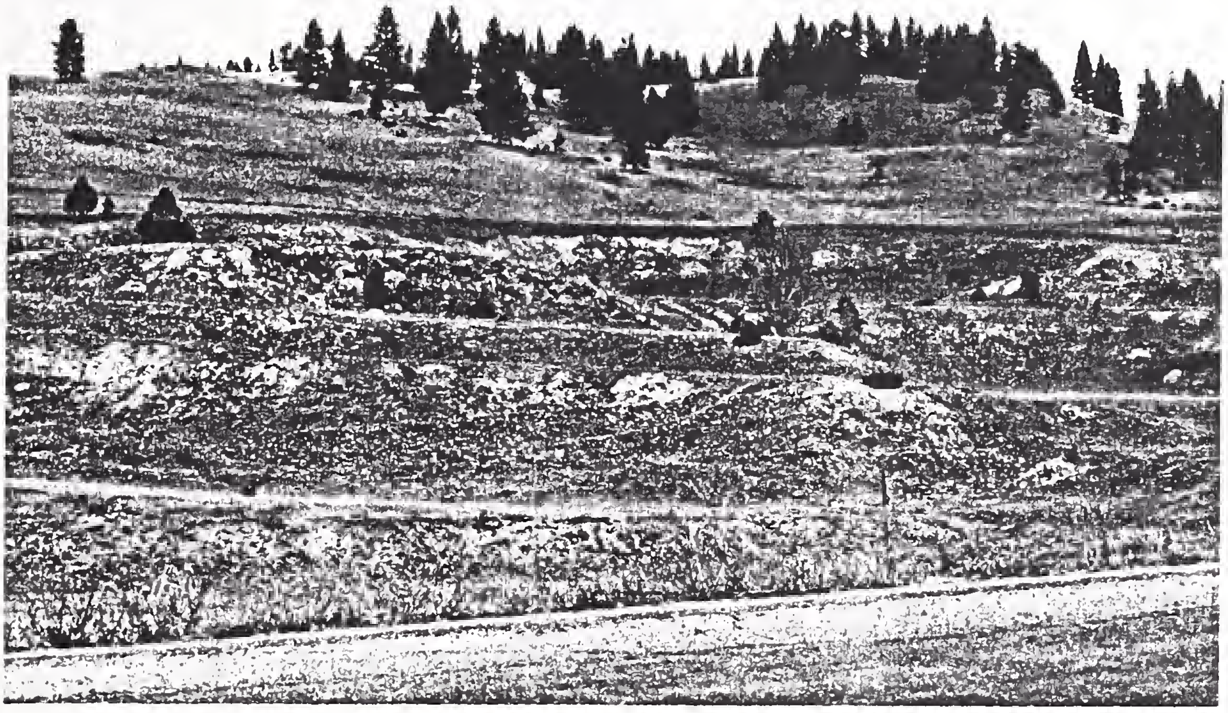


PHOTO NO. 11 - Borrow Area.
Notice the steep cut slopes have eroded and sloughed. Looking north from the county road. Located approximately 500 to 1,000 feet northwest of site.

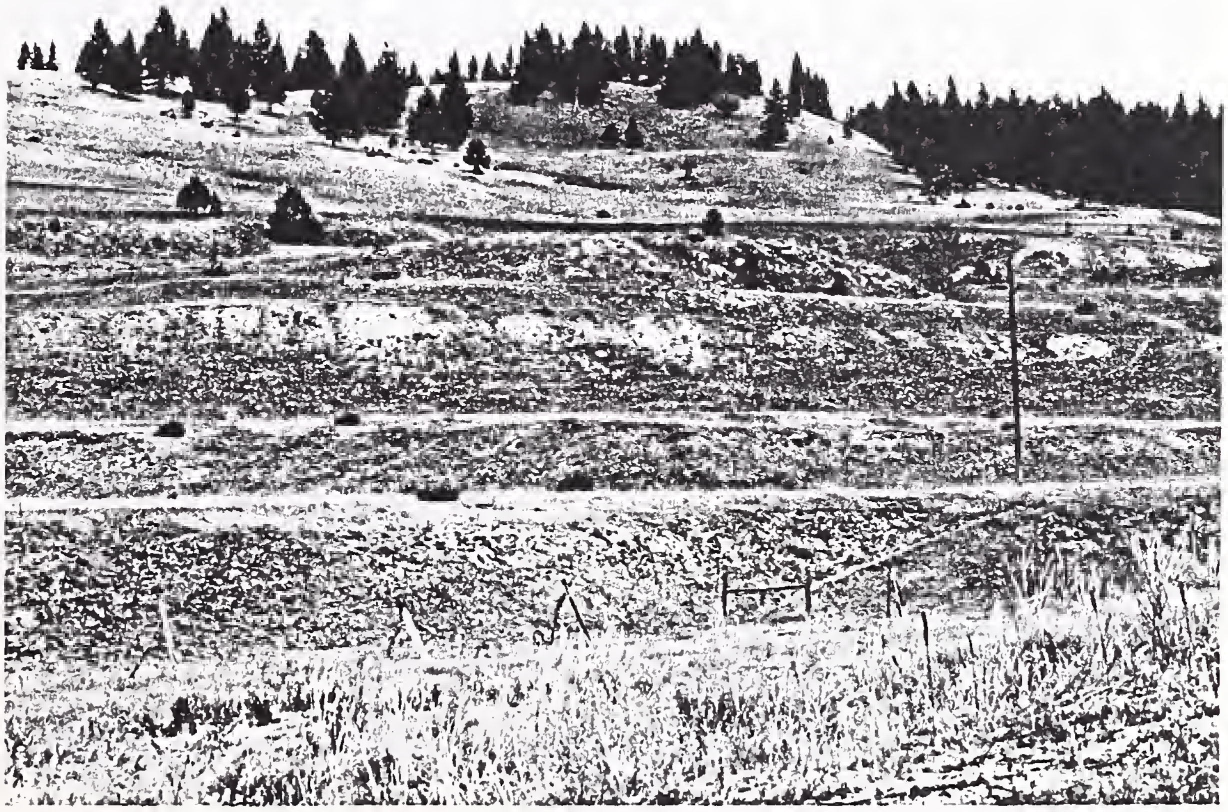


PHOTO NO. 12 - Borrow Area.
Cut slopes have been somewhat terraced. Looking northwesterly from the county road.

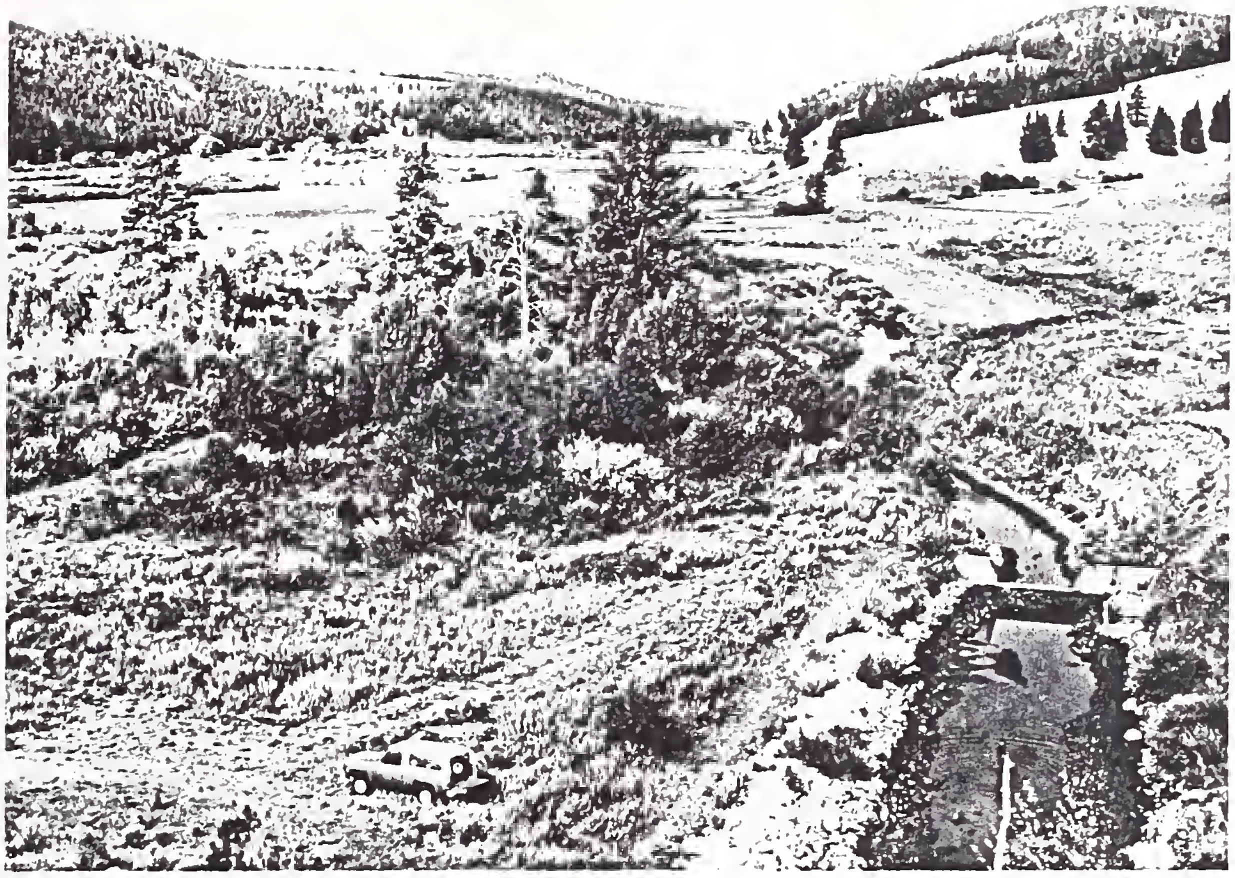


PHOTO NO. 13 - The Valley Downstream.
The outlet works return channel is shown on the right side of the photo. Watercourse shown in central-left of photo is the underseepage outlet channel.

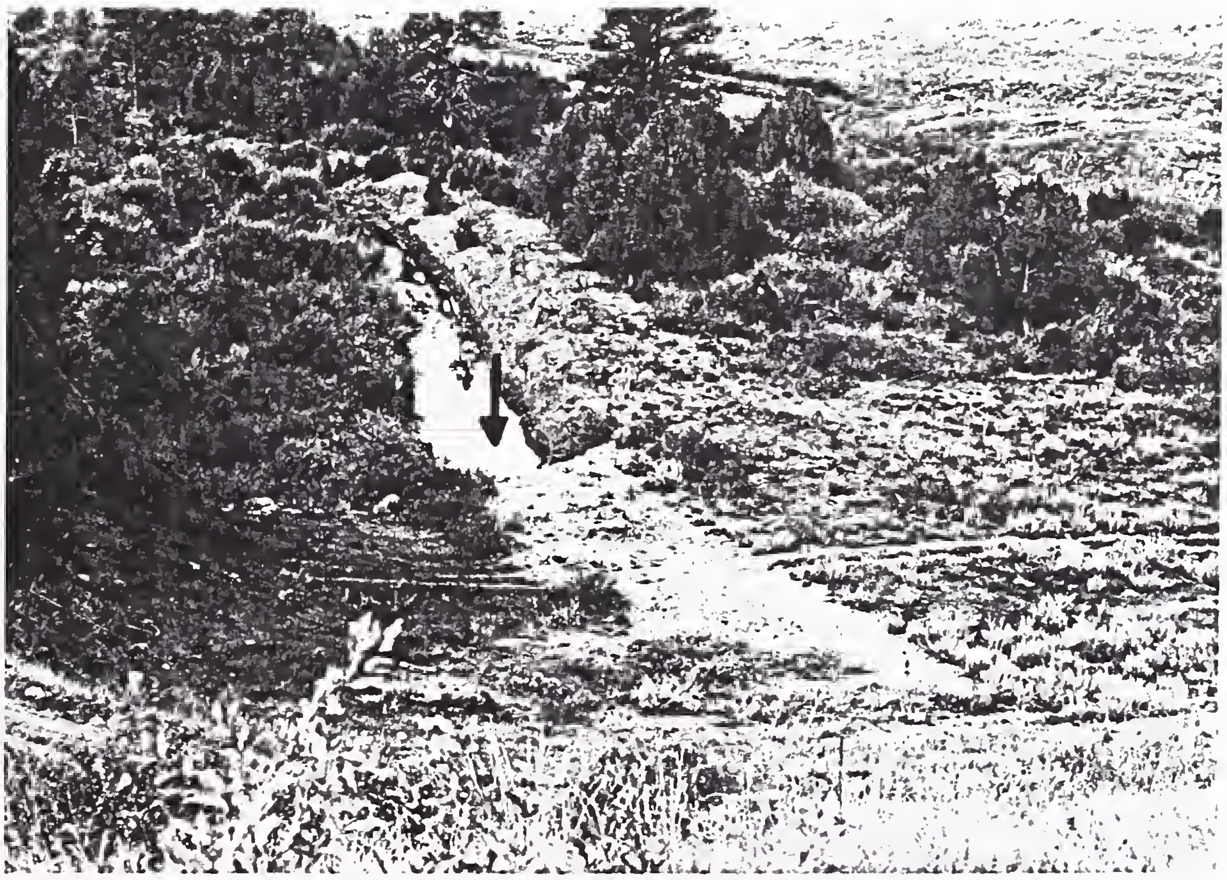


PHOTO NO. 14 - Underseepage Return Channel.
Seepage through the dam is collected in drain pipes and exists downstream. See arrow.

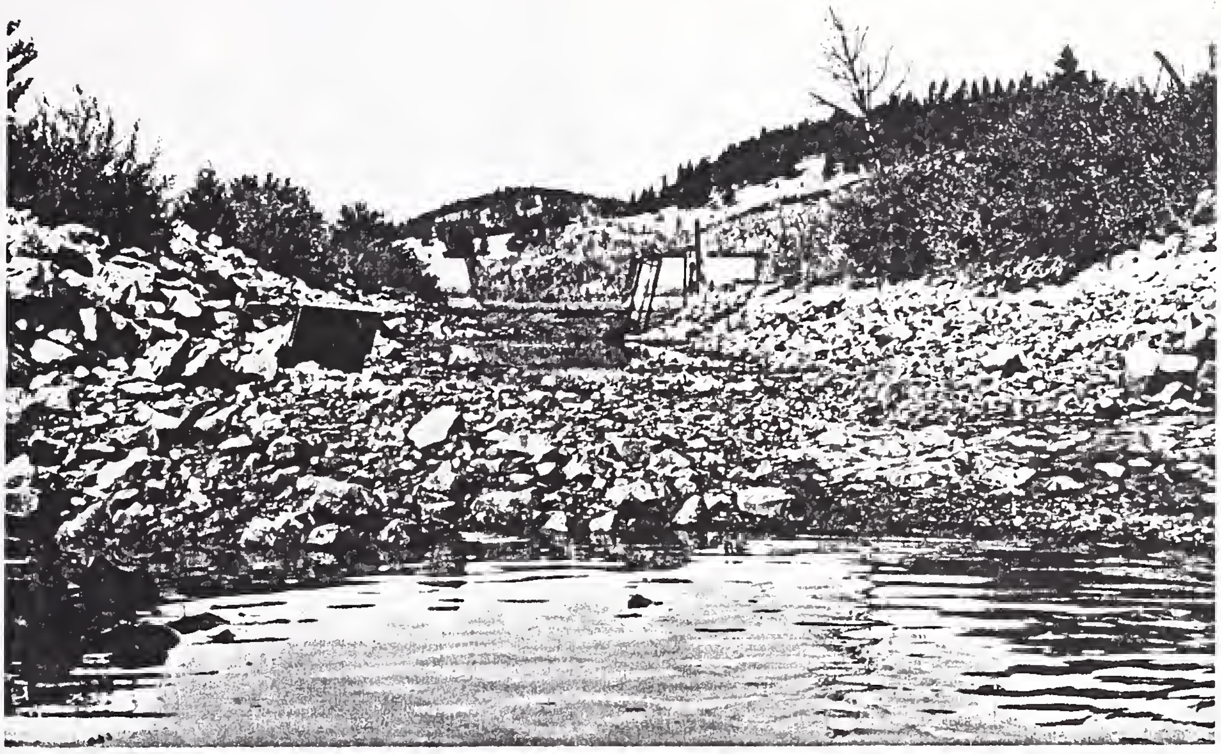


PHOTO NO. 15 - Outlet Works Stilling Basin and Return Channel. Riprap of stilling basin is in front-center of photo. Notice the right side of the stilling basin appears intact (riprap in mortar), while the left side appears as coarse loose riprap. A control weir is shown in the center of the photo.

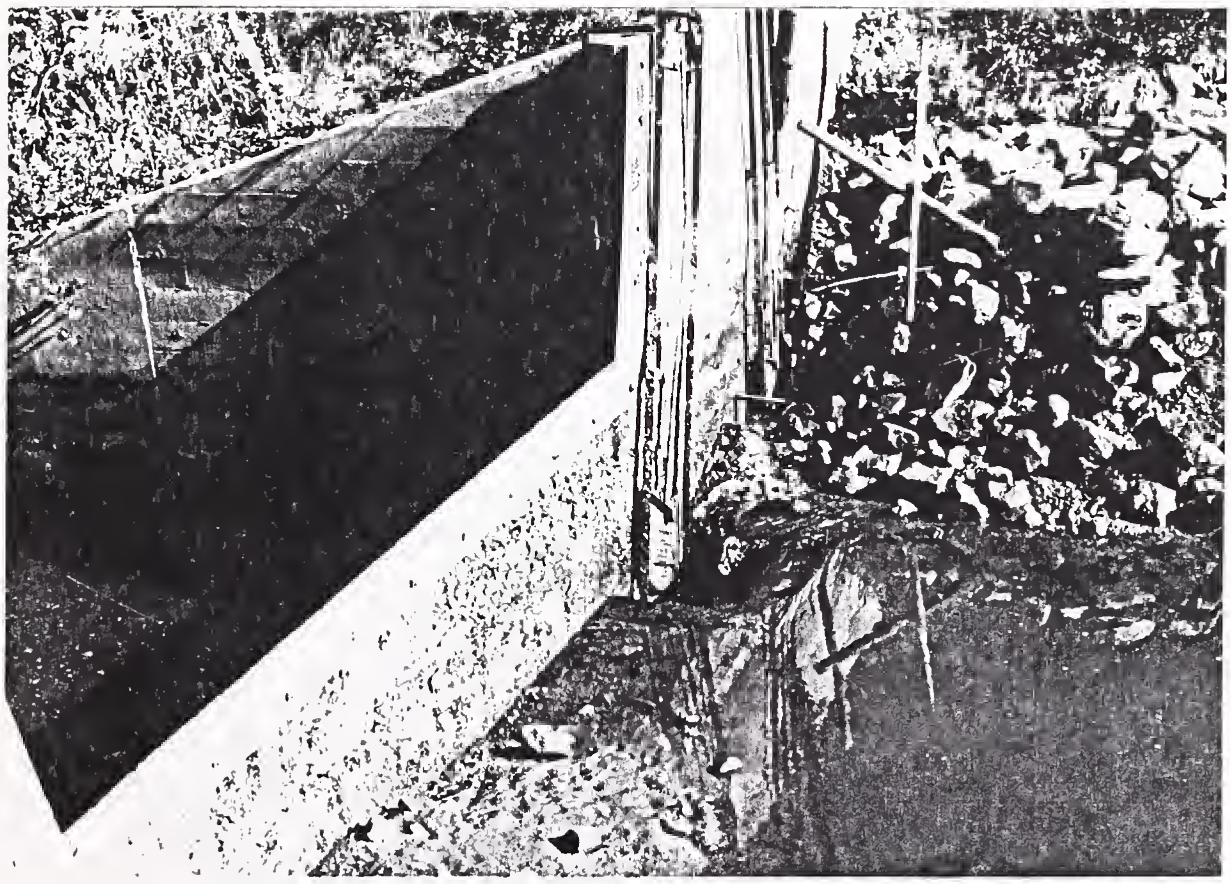


PHOTO NO. 16 - Control Weir in the Downstream Channel. The 18-inch slidegate nearest the photographer is intended for sluicing operations, but is inoperable (rusted open). Other slidegate and frame for irrigation outlet. Note concrete deterioration on upstream face of the weir structure.

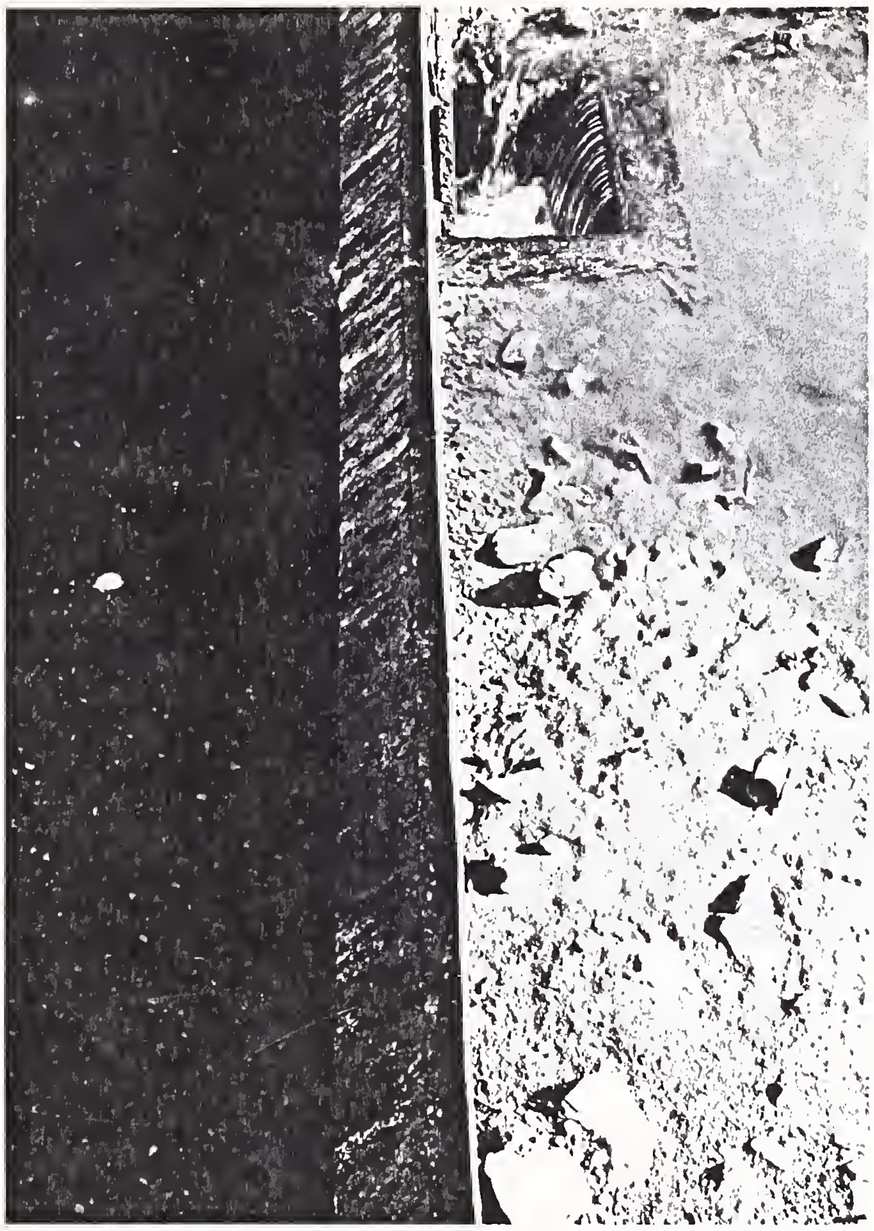


PHOTO NO. 17 - Weir Crest and Sluice Outlet.

Looking down on the control weir and the sluice outlet box. Notice the siltation and debris in front (right side) of the weir.

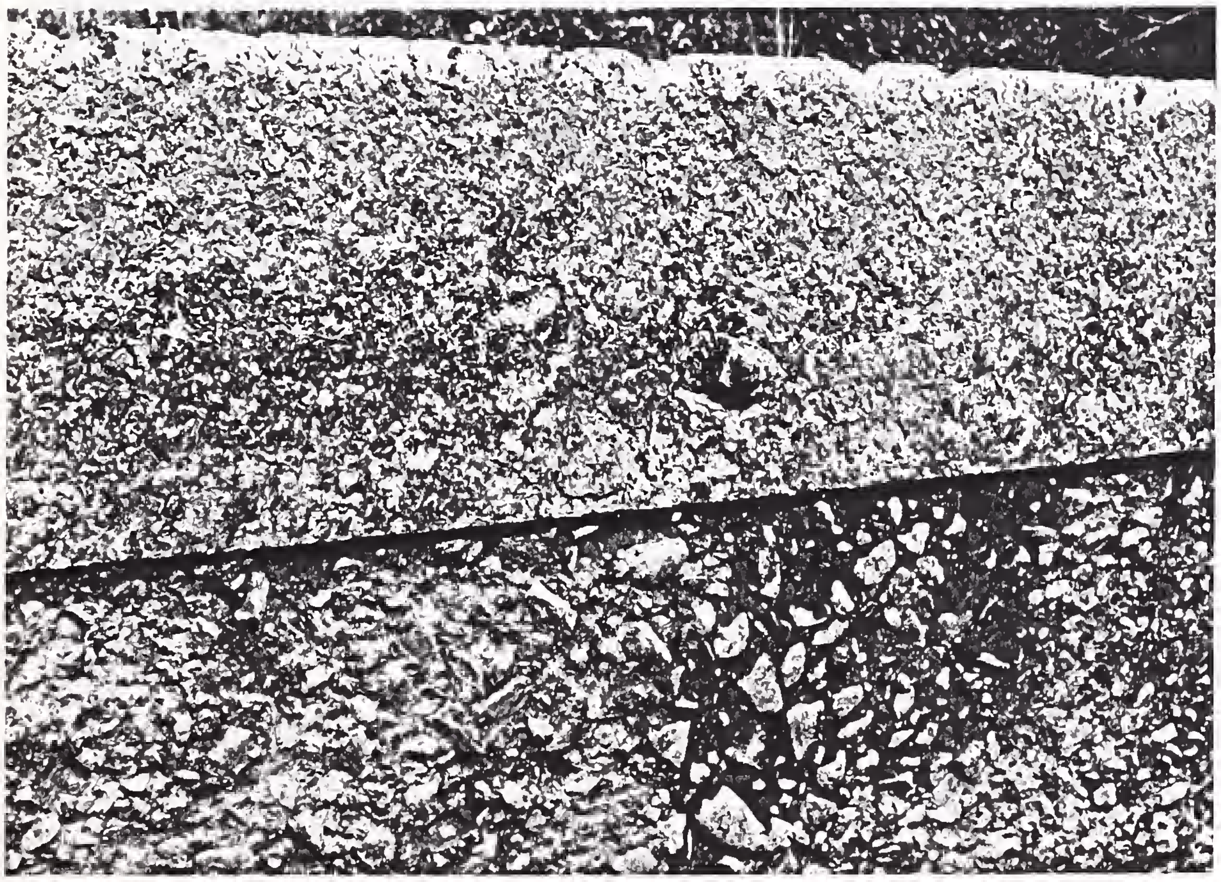


PHOTO NO. 18 - Weir Structure.

Metal plate crest of the weir is highly corroded and chipped. Concrete on the weir structure is spalling. Notice the exposed aggregate below the plate.

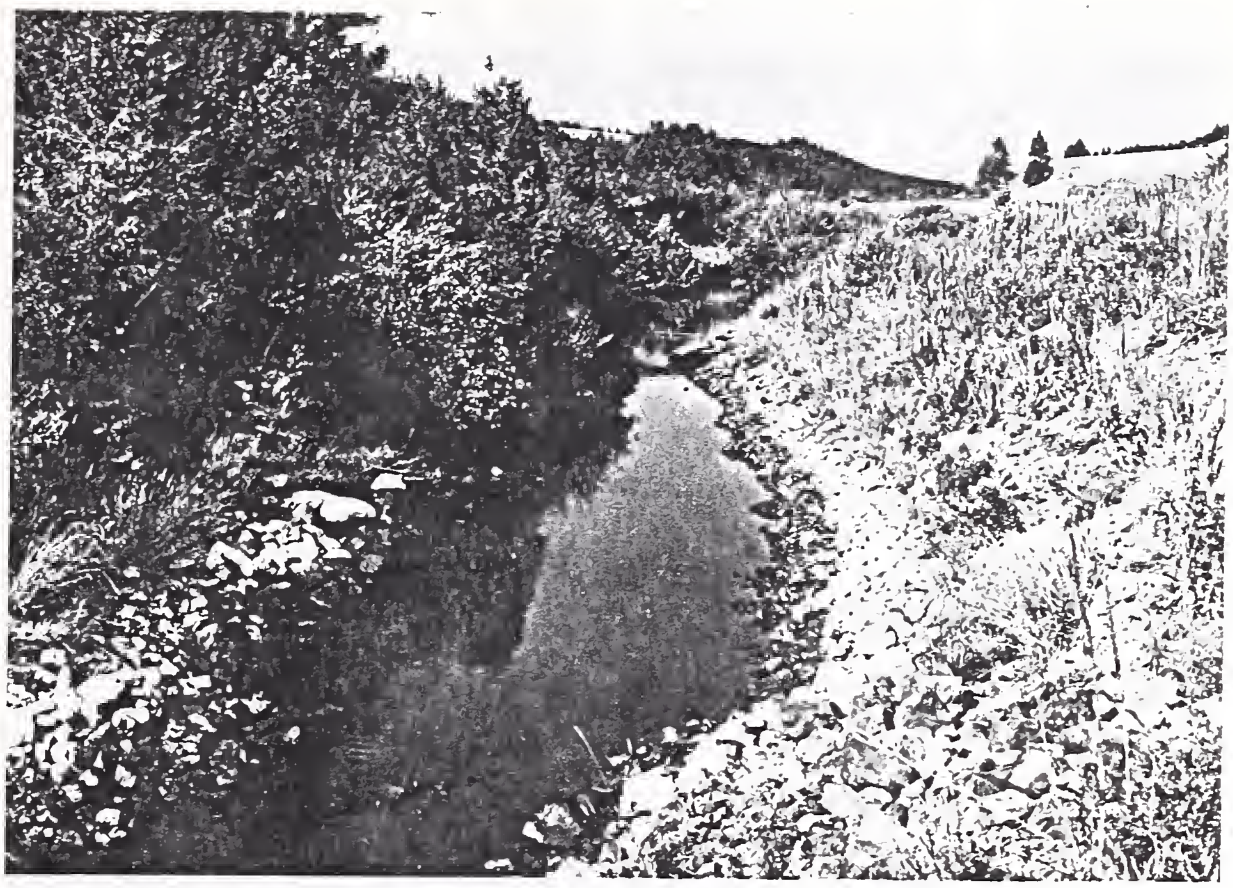


PHOTO NO. 19 - Outlet Works Return Channel.
Looking downstream from the control weir.

PHOTO NO. 20 - Outlet Works Return Channel.
Looking upstream toward the outlet and stilling pool.
Notice riprapped channel banks.

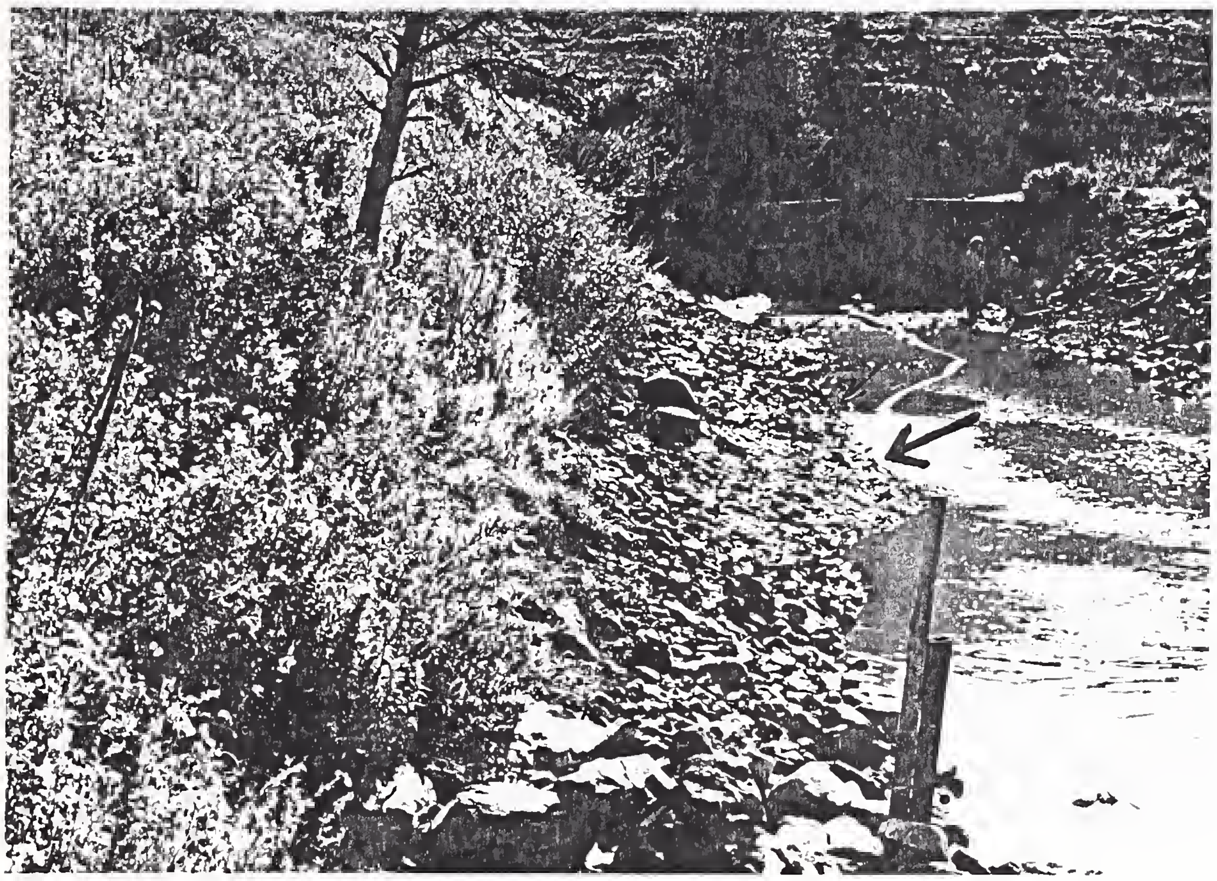


PHOTO NO. 21 - Outlet Works Return Channel.
The outlet channel has experienced disconfiguration due to a small bank failure. The toe of the failure slope extends onto the channel floor. The arrow points to the toe. Notice the staff gage in the forefront.

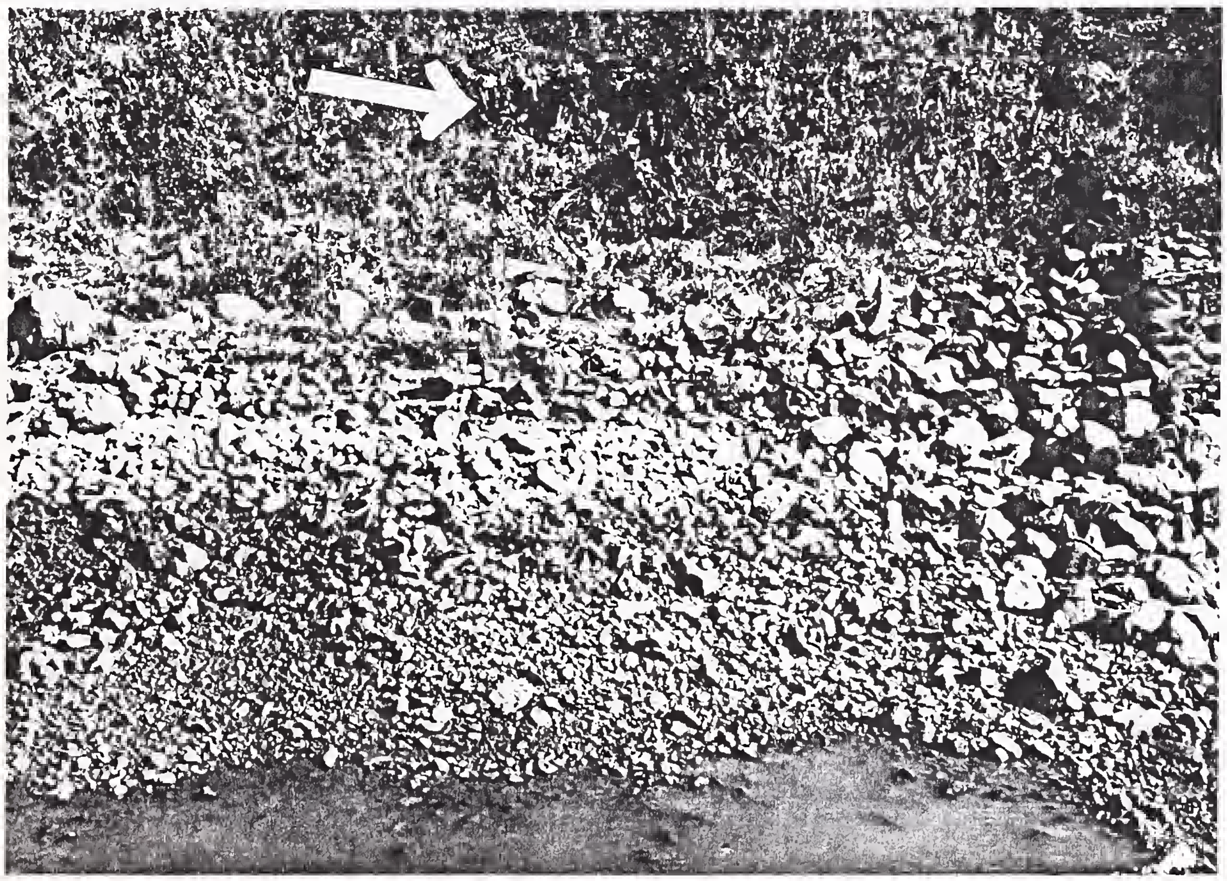


PHOTO NO. 22 - Bank Failure in Outlet Works Return Channel.
The top of the slope failure is identified by the scarp. The scarp is approximately 18 inches high and 40 feet long (See arrow).

PHOTO NO. 23 - Subsidence of Riprap in the Outlet Works Return Channel. Near the top of the channel bank the riprap has fallen as a result of the slope failure. At the toe the channel has raised. See Photo No. 21.

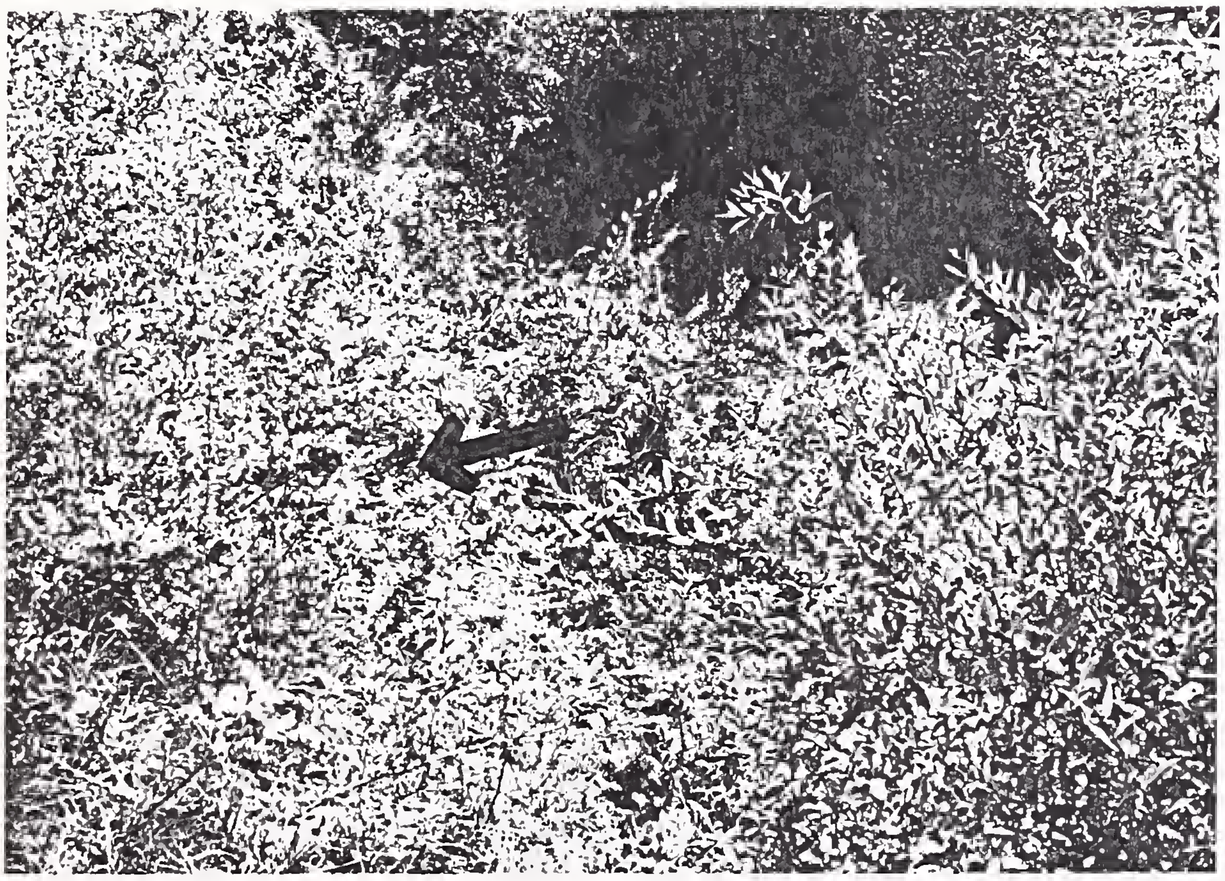


PHOTO NO. 24 - Springs.
Numerous springs exist near the top of the outlet works return channel bank failure (right side of channel below county road). The springs are probably the cause of the slope failure (see arrow).

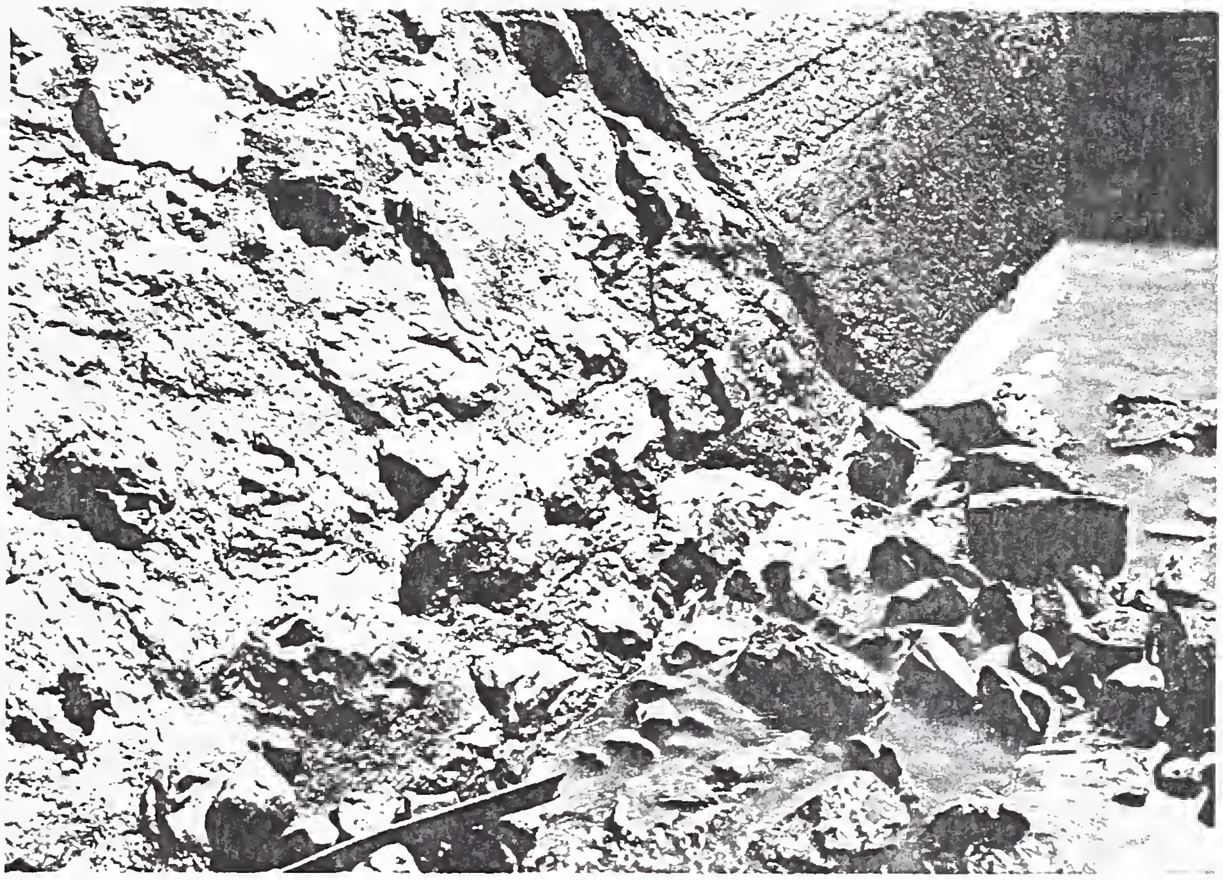


PHOTO NO. 25 - Seepage in the Outlet Works Return Channel. Hydrostatic pressure is evidenced by this trajectory of water immediately downstream of the outlet conduit. It is likely that this water is from springs on the right abutment.

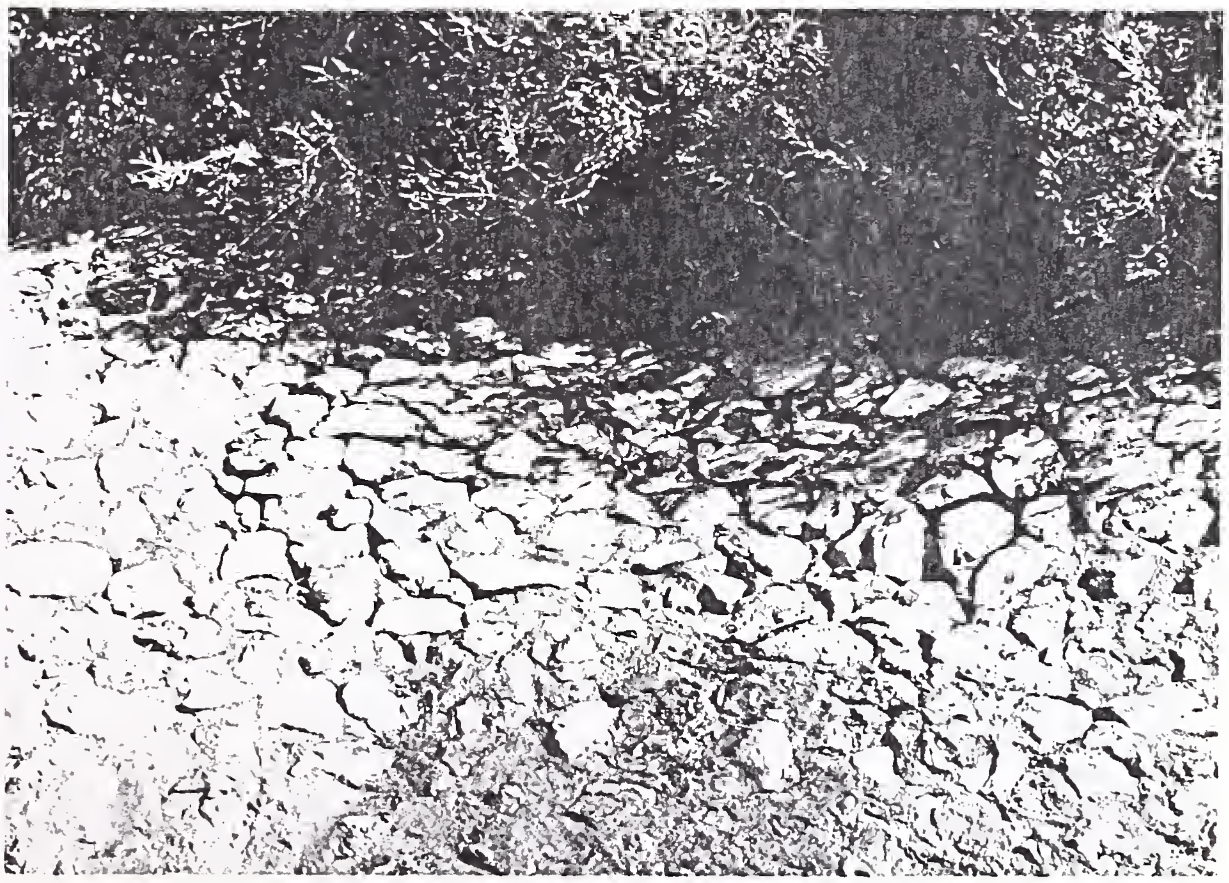


PHOTO NO. 26 - Outlet Works Return Channel Subsidence. Subsidence probably caused by spring activity. The water exiting on the channel wall in this photo has carried fine grained materials out from behind the riprap. As a result, the riprap has subsided creating a depression.

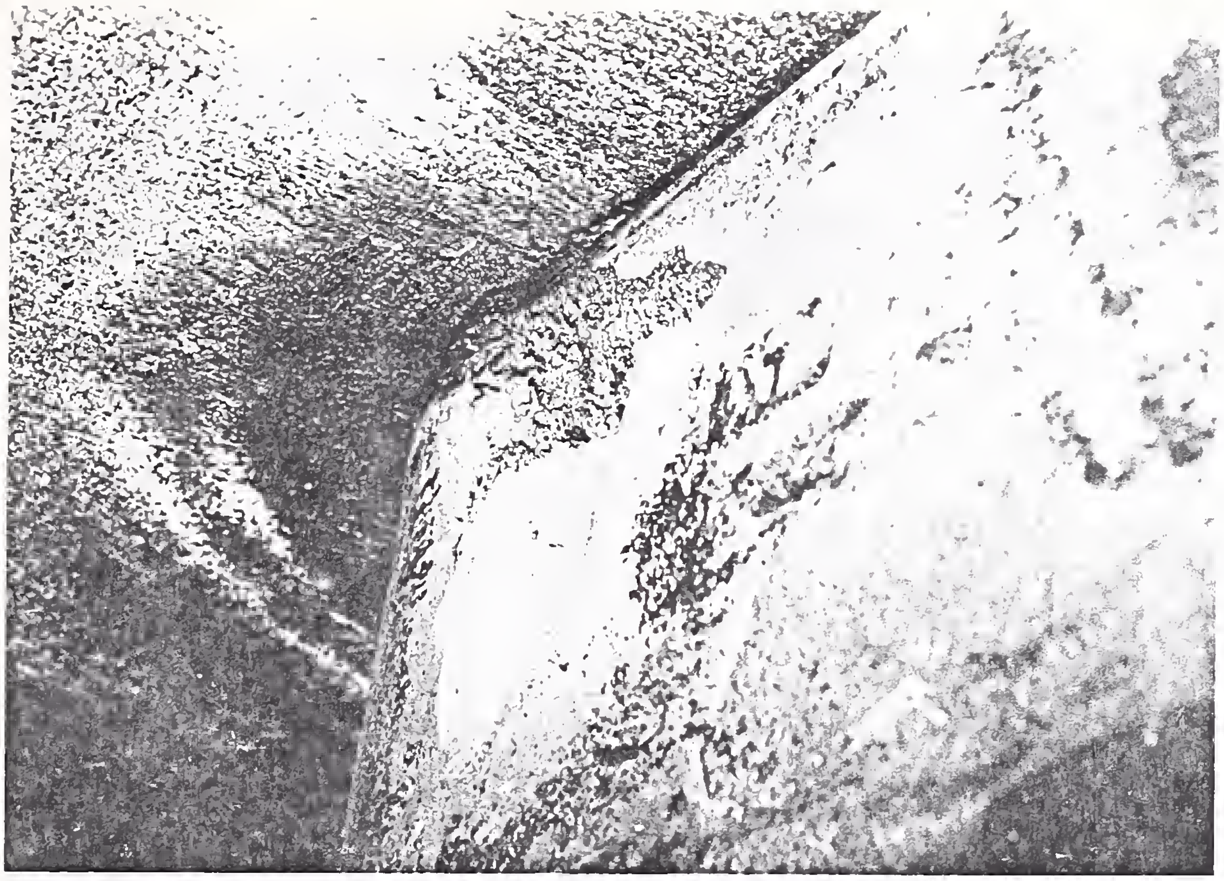


PHOTO NO. 27 - Deflector Plate on Butterfly Valve.
Pitting in the steel deflection plate of the operating gate (butterfly valve) near the top of the conduit. Some past patching can be identified. Similar pitting has occurred on the other side of the deflection plate.

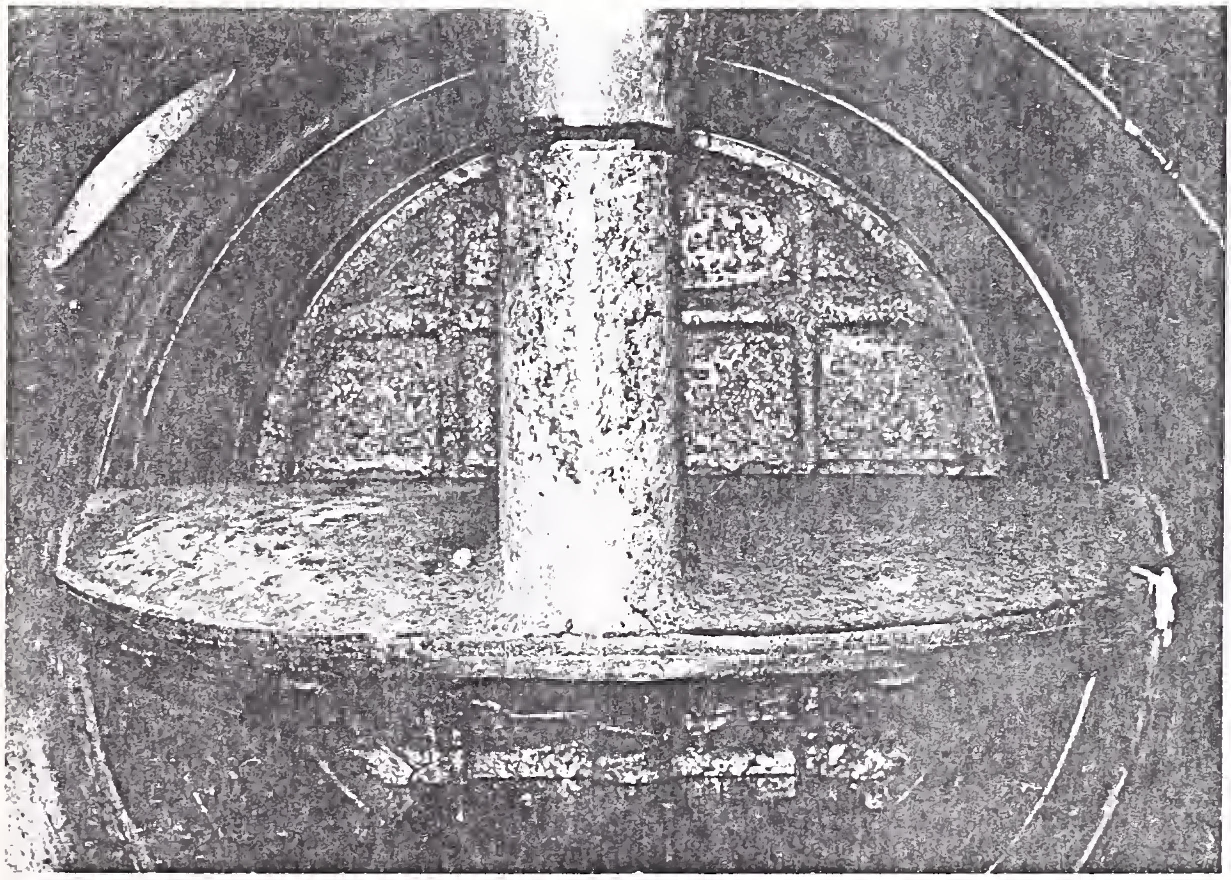


PHOTO NO. 28 - Outlet Control Gates.
The butterfly gate in the foreground is the principal control gate. The slide gate in the background is the emergency gate. Both are 54-inch diameter valves.

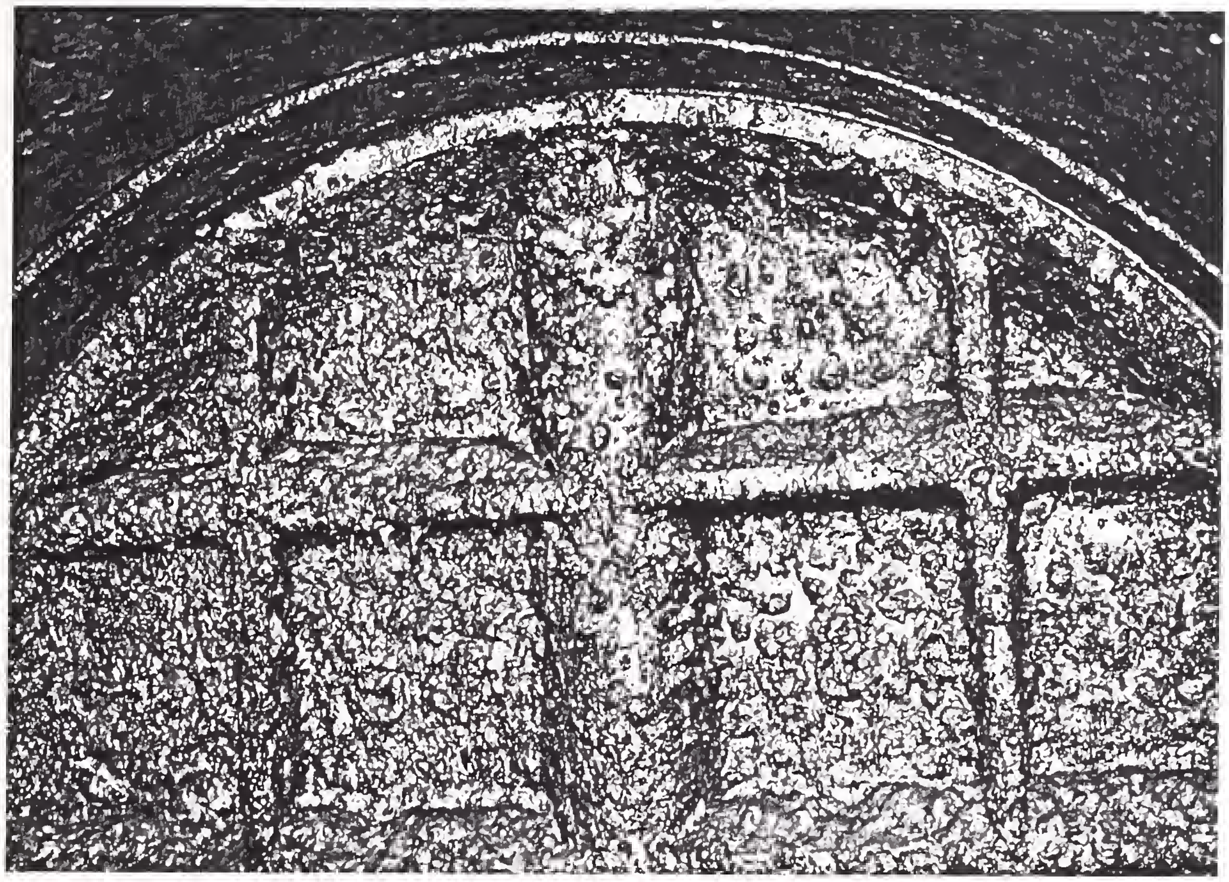


PHOTO NO. 29 - Emergency Outlet Gate.
The photo shows the emergency gate has experienced moderate corrosion. However, the gate is not leaking significantly and is functioning adequately.

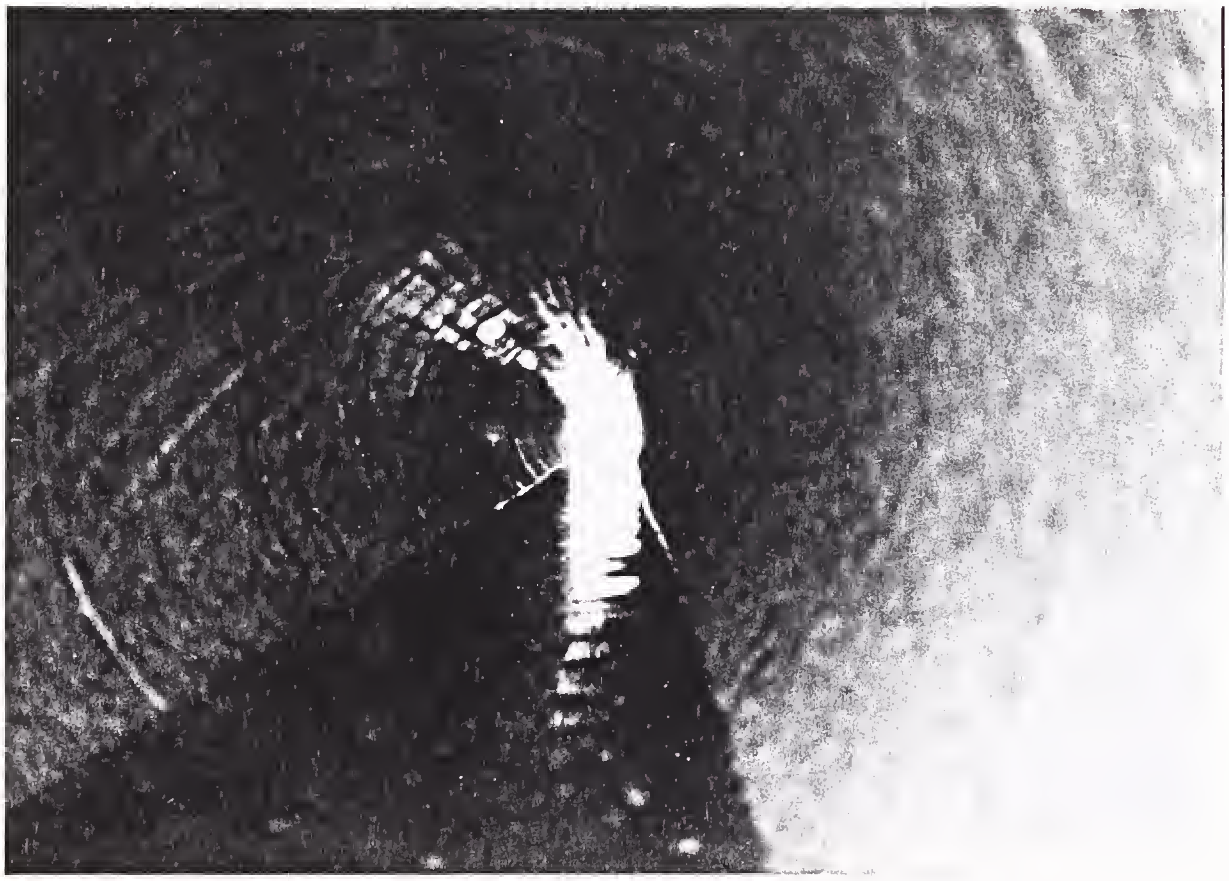


PHOTO NO. 30 - The Outlet Conduit.
The 5-foot diameter conduit is concrete. It is relatively horizontal in grade and has experienced very little differential settlement. Picture taken from inside looking out.

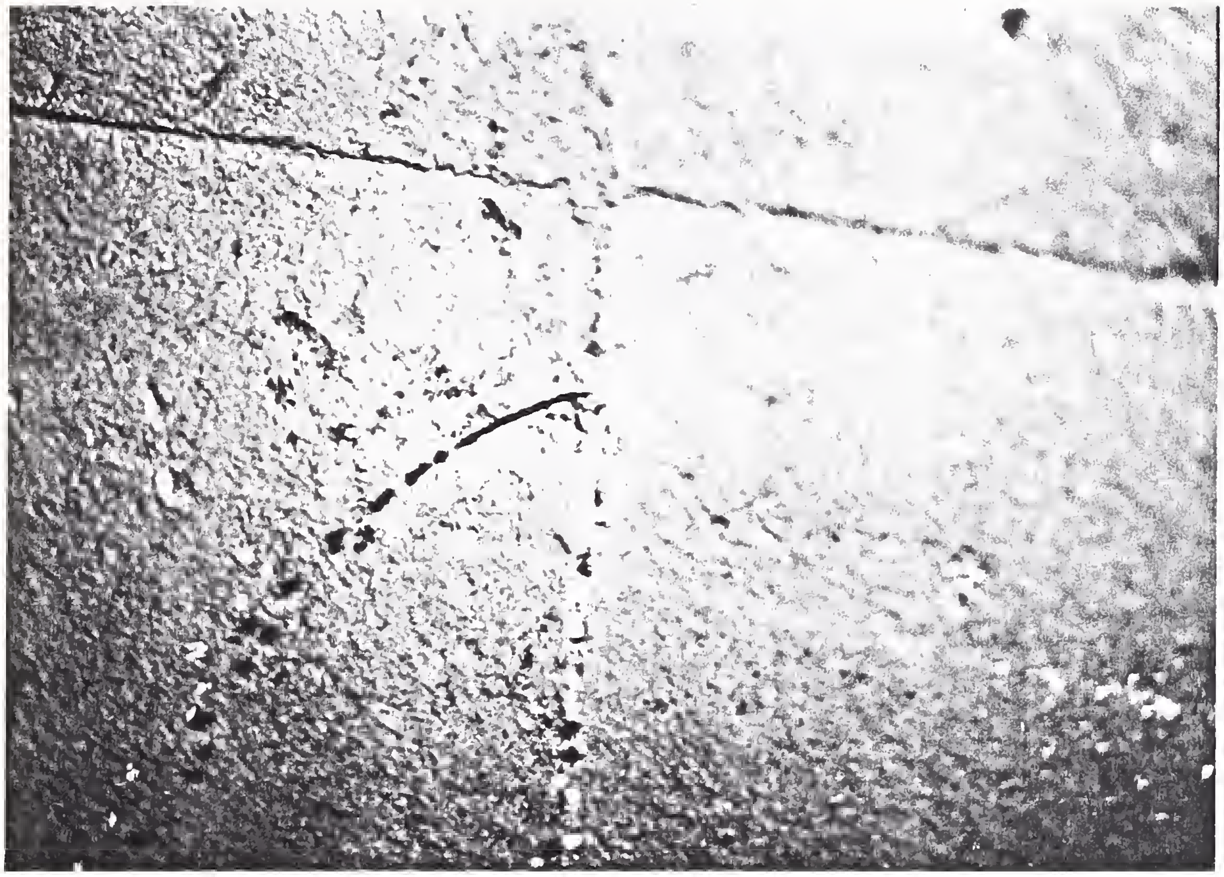


PHOTO NO. 31 - The Outlet Conduit.
There are two small leaks in the outlet conduit. The trajectory of water from a hole at a concrete joint indicates there is a significant amount of pressure exterior to the pipe.

PHOTO NO. 32 - Approach Channel to the Emergency Spillway.
Notice the dense willow growth, and concrete deterioration along the side walls.

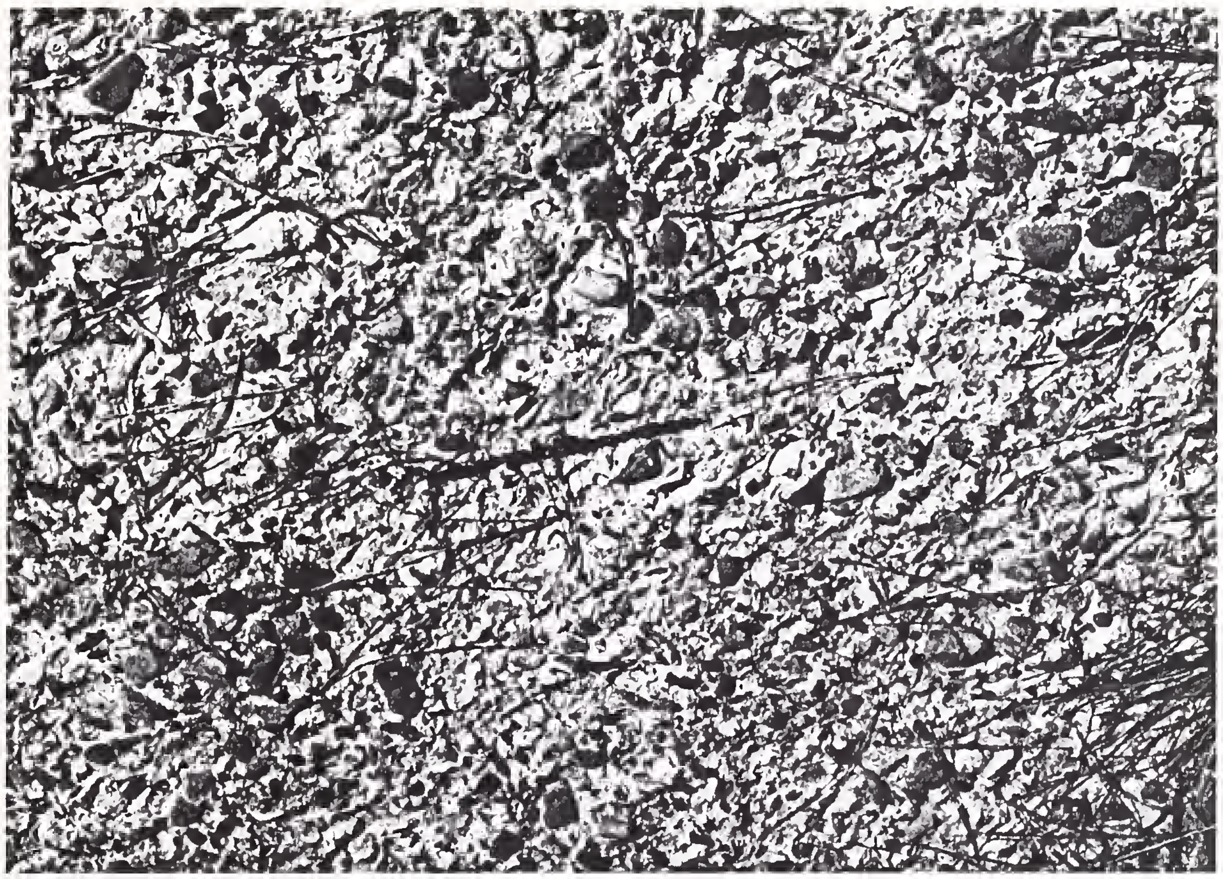


PHOTO NO. 33 - Emergency Spillway Concrete Deterioration.
This photo shows a section of the floor of the emergency spillway concrete channel. The concrete is spalling with some rebar and aggregate exposure.

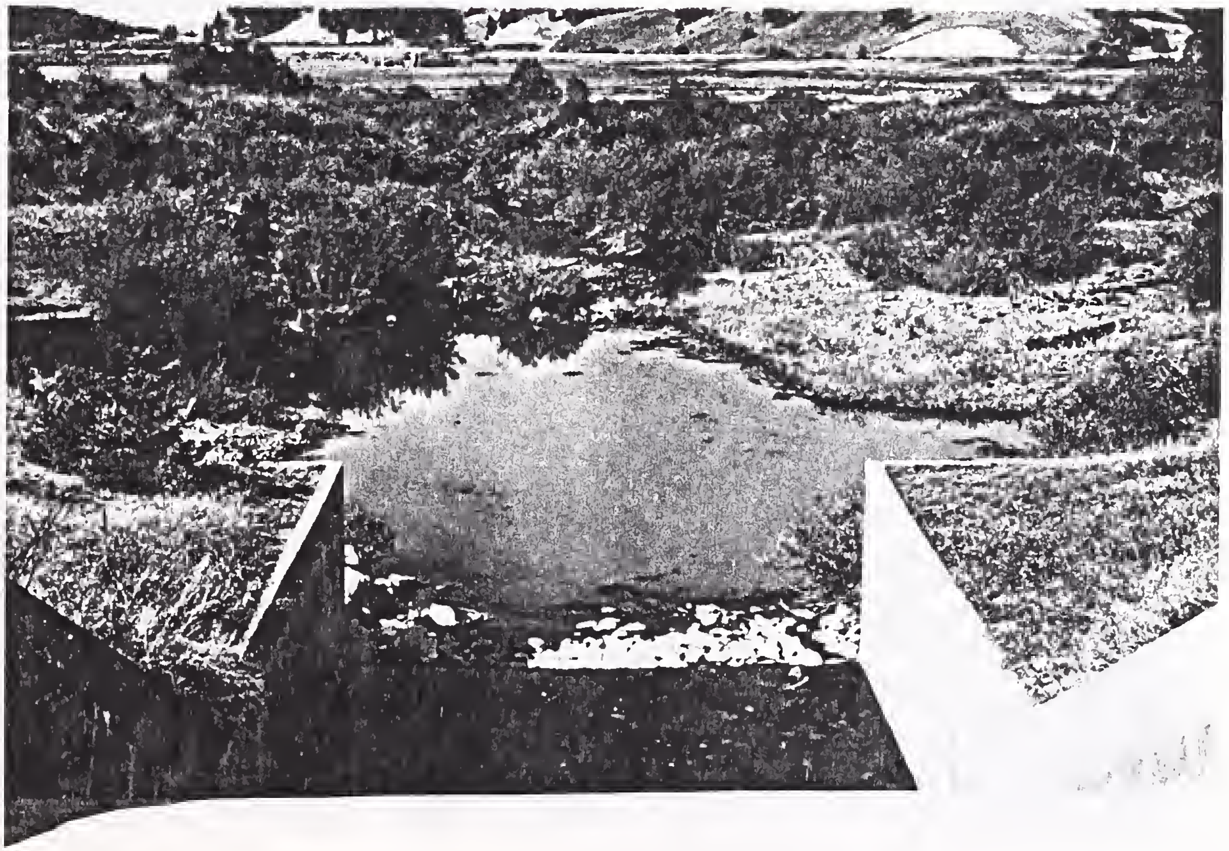


PHOTO NO. 34 - Emergency Spillway Stilling Basin.
The basin has a limited amount of riprap. Looking west from the spillway brink.

PHOTO NO. 35 - Emergency Spillway Concrete Deterioration.
Concrete spalling near the end drain.

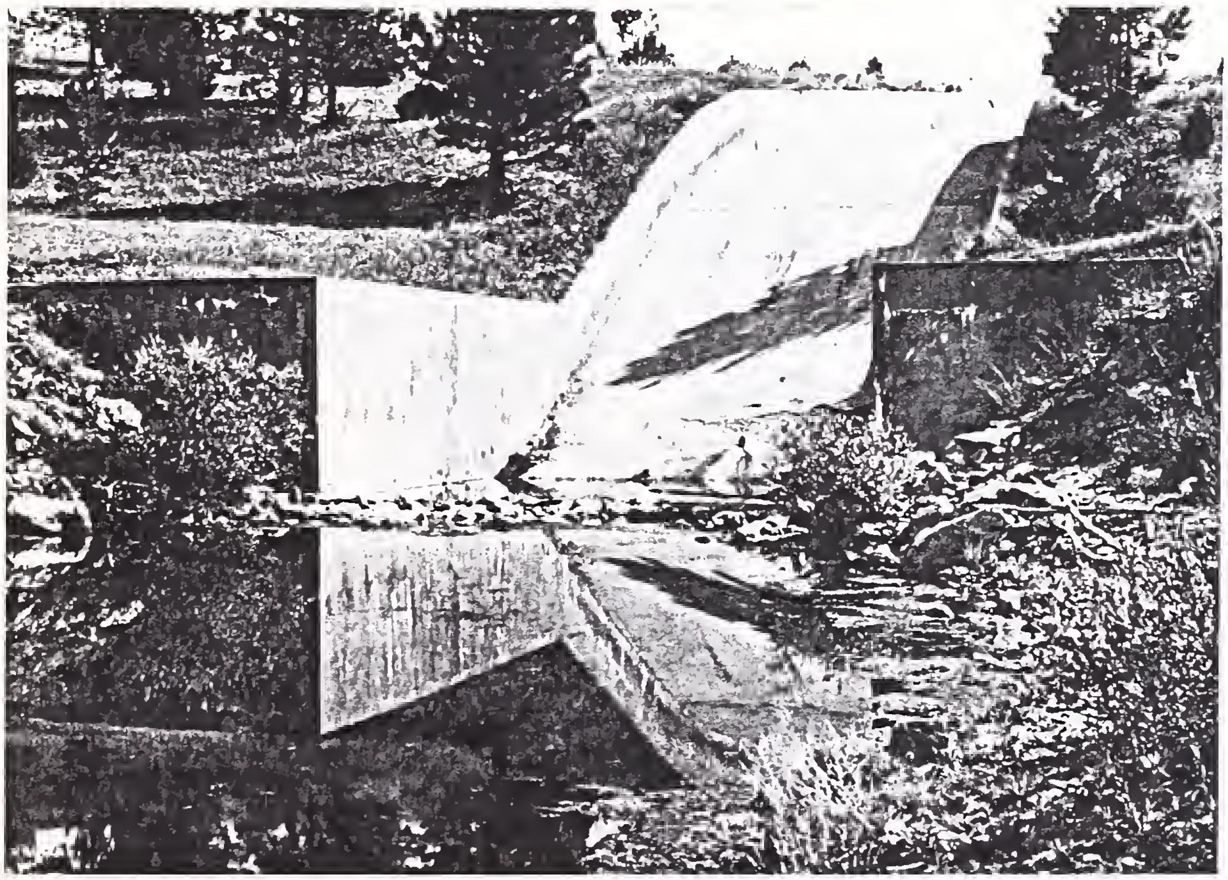


PHOTO NO. 36 - Emergency Spillway Chute and Stilling Basin.
Some riprap is evident. Spalling concrete is visible.

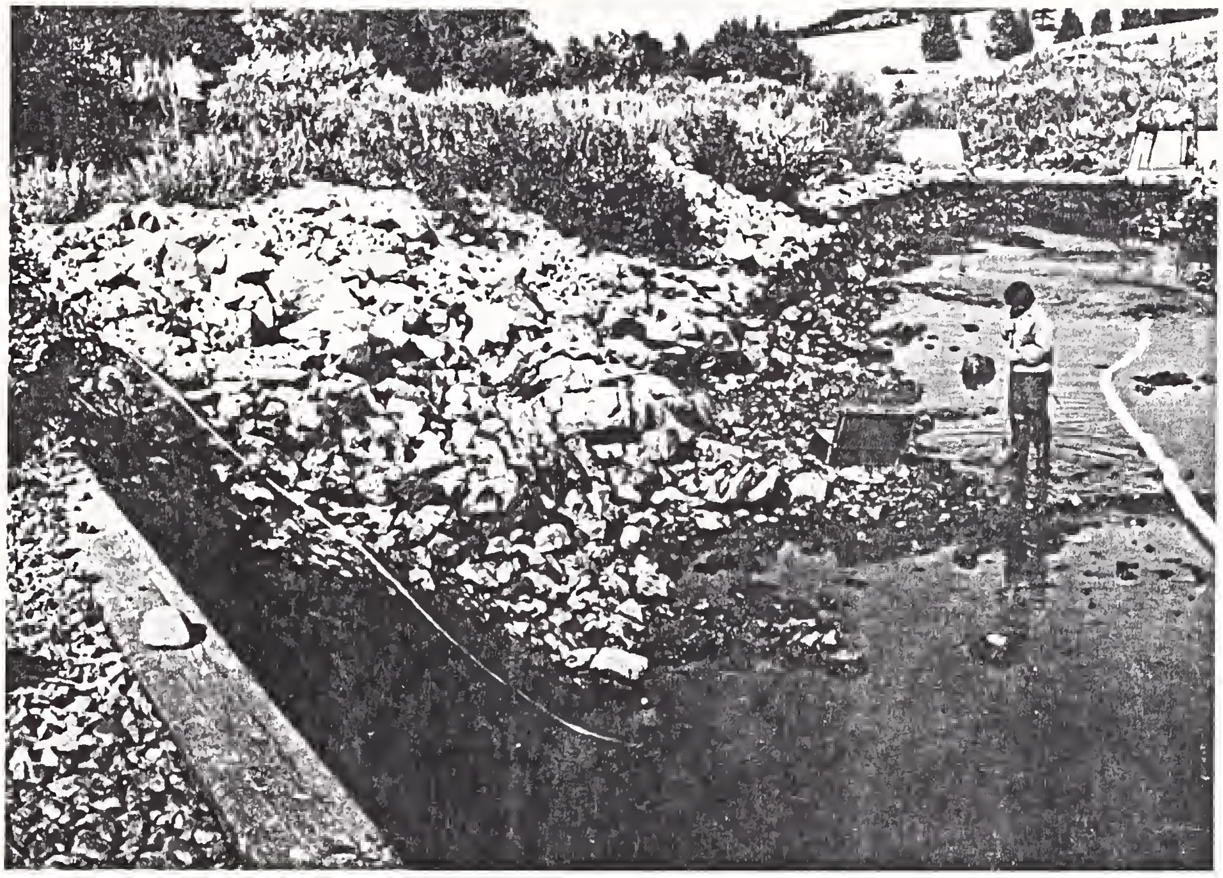


PHOTO NO. 37 - Riprap in the Outlet Works Return Channel.
The riprap material is quartzite sandstone with a mean diameter size of about 8 to 12 inches.

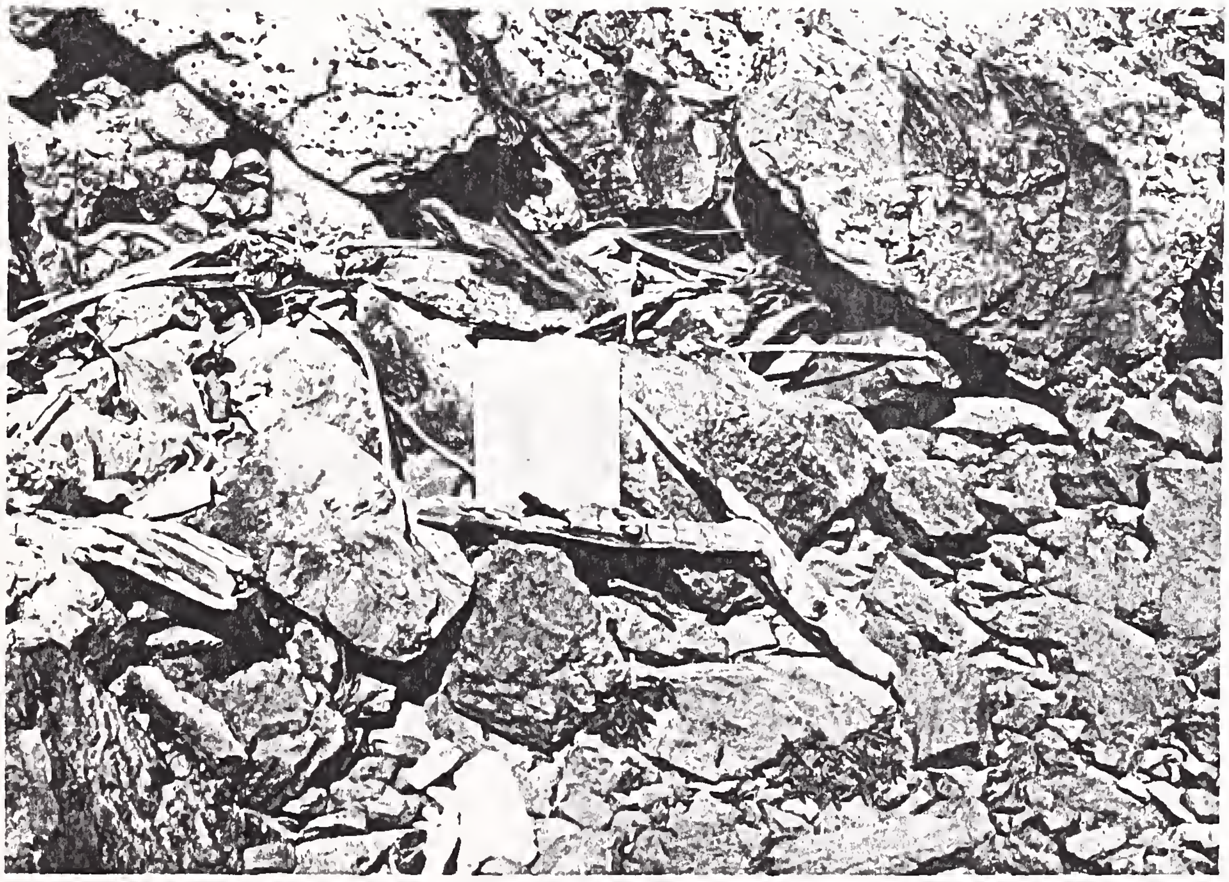
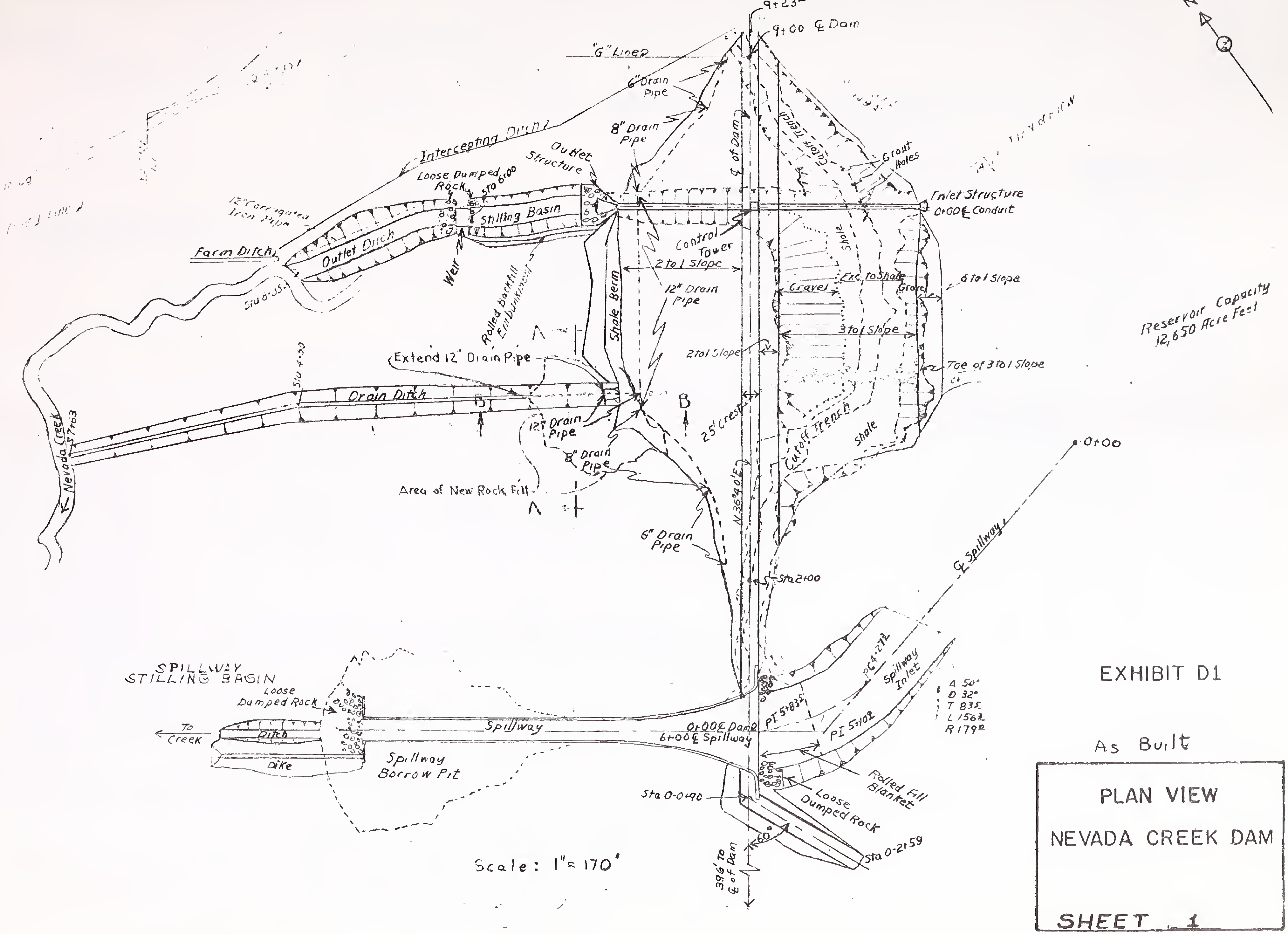


PHOTO NO. 38 - Riprap on Upstream Face of Dam.
Riprap is basically quartzite sandstone and is resistant to erosion and weathering. Notice the debris deposited during high water.

APPENDIX D

NEVADA CREEK DAM
PROJECT DRAWINGS

Exhibit D1	Construction Plans
Exhibit D2	Dam Crest Profile (Oct. 1979)
Exhibit D3	Measured Embankment Slopes (September, 1979)



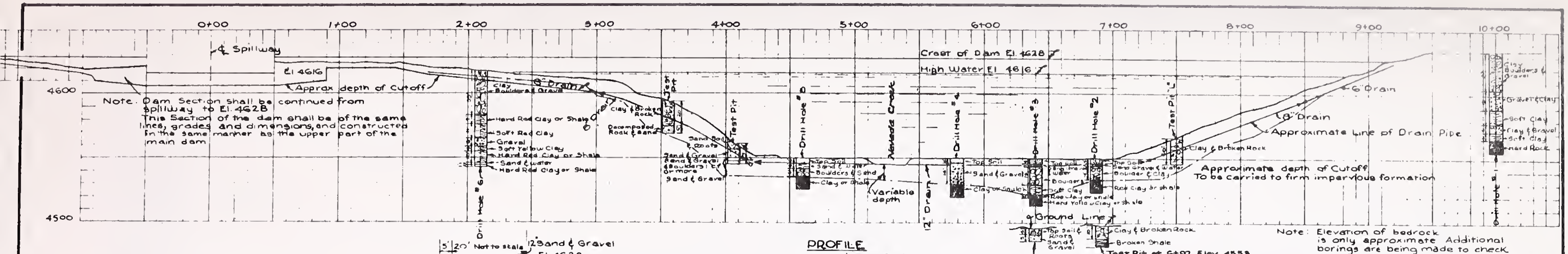
Scale: 1" = 170'

EXHIBIT D1

As Built

PLAN VIEW
NEVADA CREEK DAM

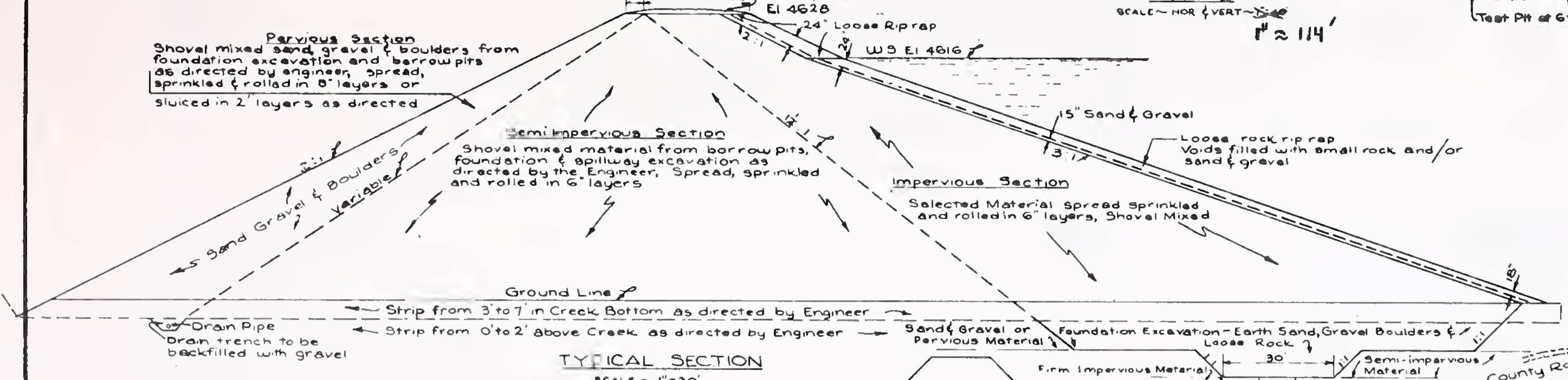
SHEET 1



Note. Dam Section shall be continued from spillway to El. 4628
This Section of the dam shall be of the same lines, grades and dimensions, and constructed in the same manner as the upper part of the main dam

PROFILE
SCALE ~ HOR & VERT ~ 1" ≈ 114'

Note: Elevation of bedrock is only approximate Additional borings are being made to check the location of bed Rock. These data will be available to the Contractor prior to the opening of bids.


5'
20'
Not to scale
12" Sand & Gravel
El 4628
24" Loose Rip rap
2:1
24"
WS El 4616
15" Sand & Gravel
3:1
Loose rock rip rap
Voids filled with small rock and/or sand & gravel
2:1
3:1
5 Sand Gravel & Boulders
Variable
18"
Ground Line
Strip from 3' to 7' in Creek Bottom as directed by Engineer
Strip from 0' to 2' above Creek as directed by Engineer
Drain Pipe
Drain trench to be backfilled with gravel
Sand & Gravel or Pervious Material
Foundation Excavation – Earth Sand, Gravel Boulders & Loose Rock
1:1
30'
Firm Impervious Material
5'
Semi-impervious Material

Pervious Section
Shovel mixed sand, gravel & boulders from foundation excavation and borrow pits as directed by engineer, spread, sprinkled & rolled in 8" layers or sluiced in 2' layers as directed

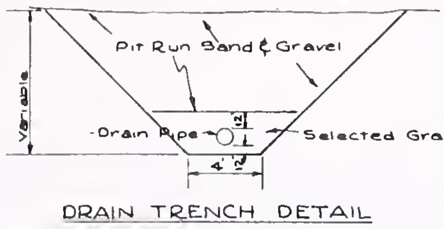
Semi Impervious Section
Shovel mixed material from borrow pits, foundation & spillway excavation as directed by the Engineer, Spread, sprinkled and rolled in 6" layers

Impervious Section
Selected Material spread sprinkled and rolled in 6" layers, Shovel Mixed

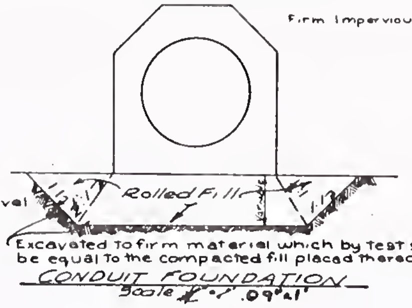
TYPICAL SECTION
SCALE ~ 1"=30'
1" ≈ 57'

CAPACITY TABLE

Contour Interval	Acres Flooded	Acre Feet Between Contours	Total Acre Feet Storage
4550	5	0	0
4555	13	40	40
4560	37	124	164
4565	72	272	436
4570	102	435	871
4575	156	645	1516
4580	195	877	2395
4585	222	1043	3436
4590	245	1167	4603
4595	269	1283	5886
4600	296	1412	7298
4605	317	1534	8832
4610	346	1658	10490
4615	367	1782	12260
4618	394	1140	13412
4625	448	2947	16359
4630			

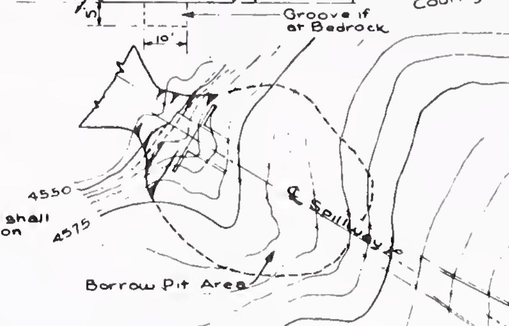


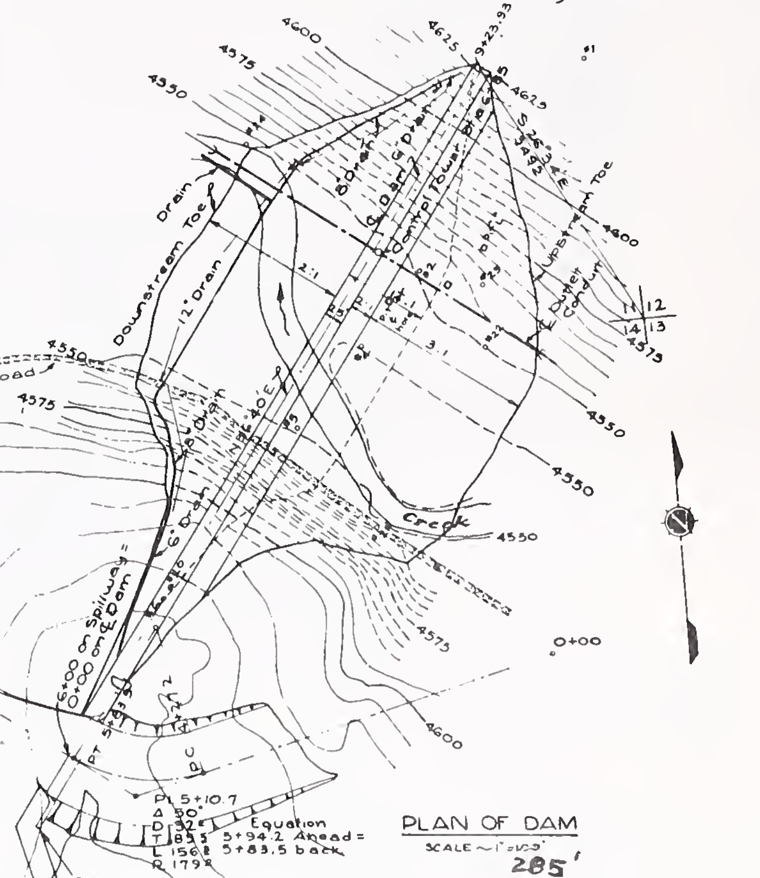
DRAIN TRENCH DETAIL



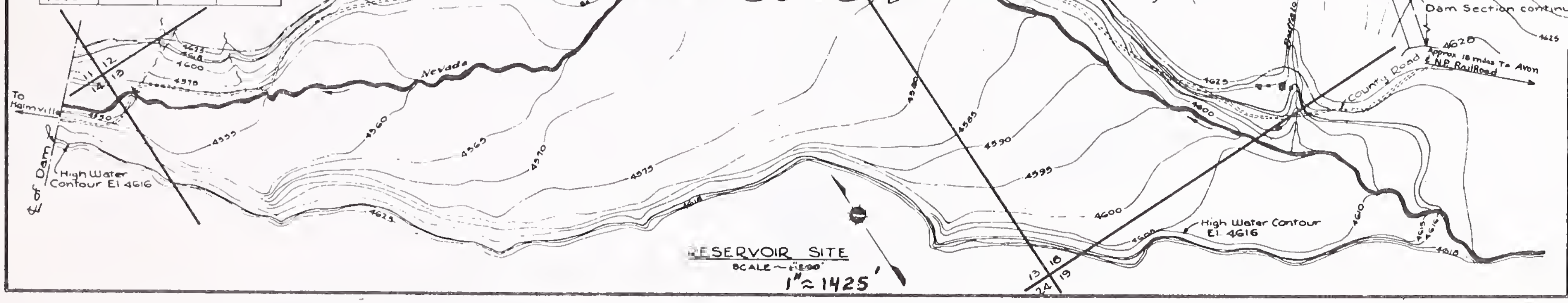
Excavated to firm material which by test shall be equal to the compacted fill placed thereon

CONDUIT FOUNDATION
Scale 1"=1'.09"=1'





PLAN OF DAM
SCALE ~ 1"=200'
285'



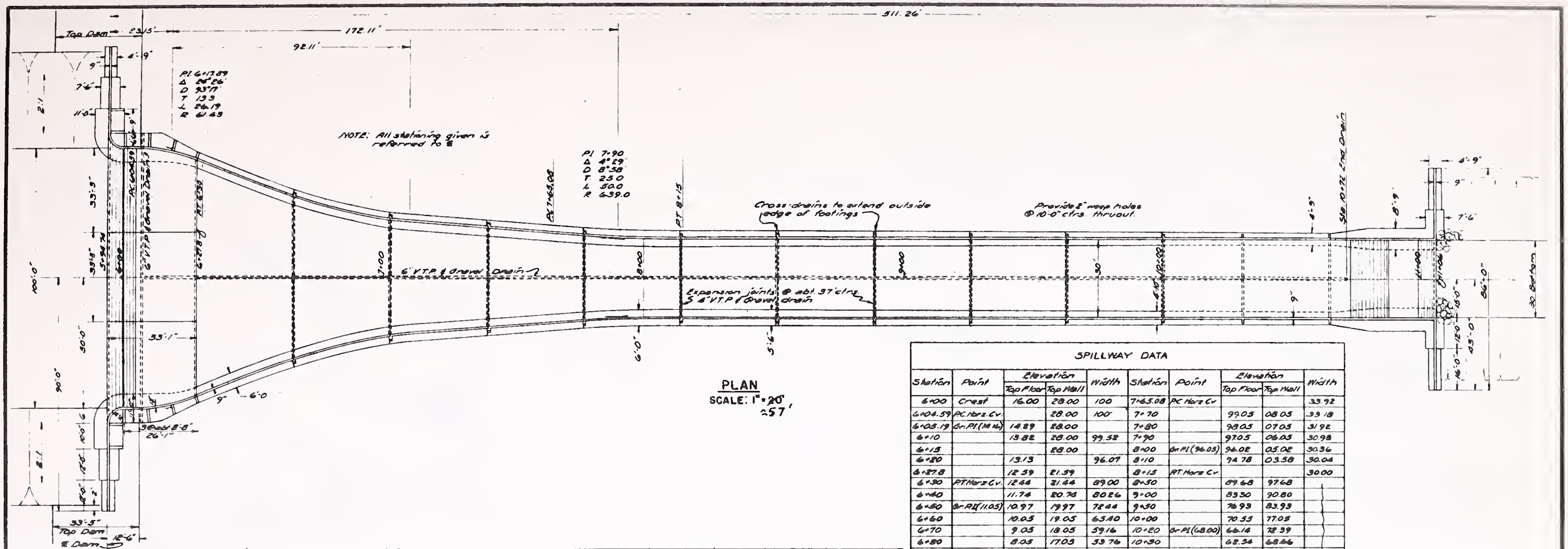
RESERVOIR SITE
SCALE ~ 1"=1200'
1" ≈ 1425'

DETAILS OF DAM
NEVADA CREEK STORAGE PROJECT
POWELL COUNTY

DEPARTMENT OF NATURAL RESOURCES
AND CONSERVATION
WATER RESOURCES DIVISION, ENGINEERING BUREAU

DESIGNED BY ______ DATE ______
DRAWN BY JCK & Traced DATE Oct 1937
APPROVED BY ______ TITLE ______ DATE ______
SHEET NO ______ OF ______
Traced by JPS Sept 1977
Dwg #1
#5-182-3

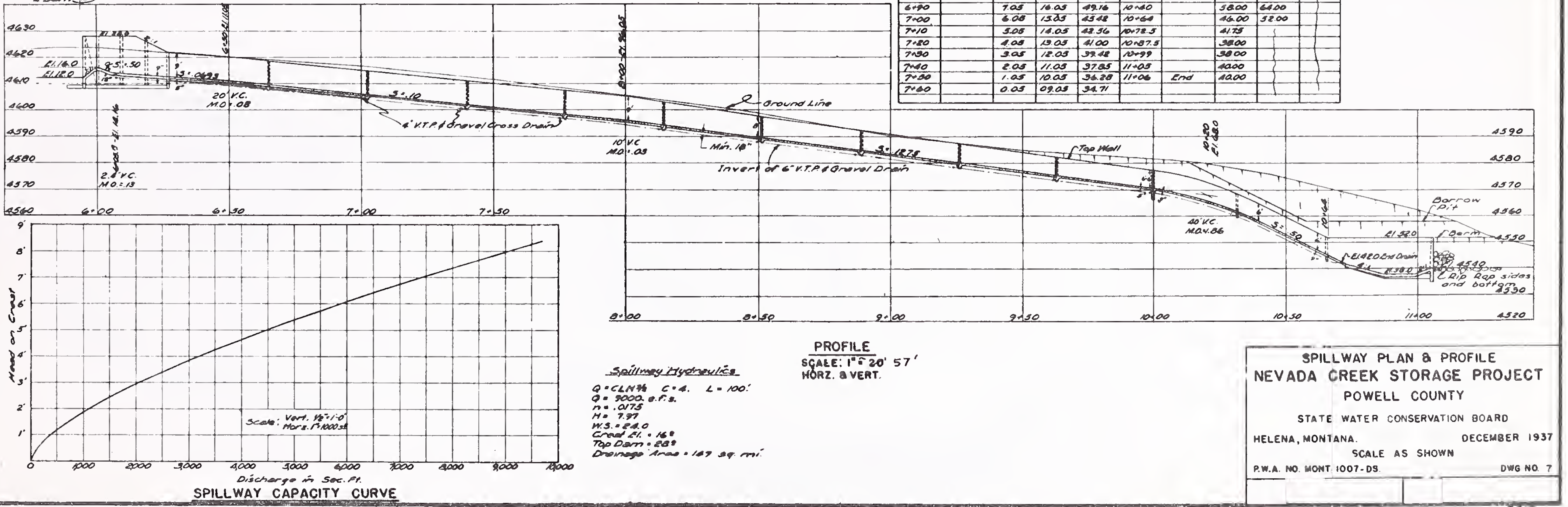
SHEET 2



PLAN
SCALE: 1" = ~~20'~~ 57'

Station	Point	Elevation		Width	Station	Point	Elevation		Width
		Top Floor	Top Wall				Top Floor	Top Wall	
6+00	Crest	16.00	28.00	100	7+65.08	PC Horz Cv			33.92
6+04.59	P.C. Horz. Cv.		28.00	100	7+70		99.05	08.05	33.18
6+05.19	Gr. P.I. (14.89)	14.89	28.00		7+80		98.05	07.05	31.92
6+10		13.82	28.00	99.52	7+90		97.05	06.05	30.98
6+15			28.00		8+00	Gr. P.I. (96.05)	96.02	05.02	30.36
6+20		13.13		96.07	8+10		94.78	03.58	30.04
6+27.8		12.59	21.59		8+15	P.T. Horz Cv.			30.00
6+30	P.T. Horz. Cv.	12.44	21.44	89.00	8+50		89.68	97.68	
6+40		11.74	20.74	80.26	9+00		83.30	90.80	
6+50	Gr. P.I. (11.05)	10.97	19.97	78.44	9+50		76.93	83.93	
6+60		10.05	19.05	65.40	10+00		70.55	77.05	
6+70		9.05	18.05	59.16	10+20	Gr. P.I. (68.00)	66.14	72.39	
6+80		8.05	17.05	53.76	10+30		62.54	68.66	
6+90		7.05	16.05	49.16	10+40		58.00	64.00	
7+00		6.08	15.05	45.48	10+64		46.00	52.00	
7+10		5.08	14.05	42.56	10+72.5		41.75		
7+20		4.08	13.05	41.00	10+87.5		38.00		
7+30		3.05	12.05	39.48	10+99		38.00		
7+40		2.05	11.05	37.85	11+05		40.00		
7+50		1.05	10.05	36.28	11+06	End	40.00		
7+60		0.05	09.05	34.71					

SPILLWAY DATA



PROFILE
SCALE: 1" = 20' 57'
HORZ. & VERT.

SPILLWAY CAPACITY CURVE

Spillway Hydraulics

Q = CLH³⁄₂ C = 4. L = 100'
Q = 9000 c.f.s.
n = .0175
H = 7.97
W.S. = 24.0
Crest El. = 16°
Top Dam = 28°
Drainage Area = 147 sq. mi.

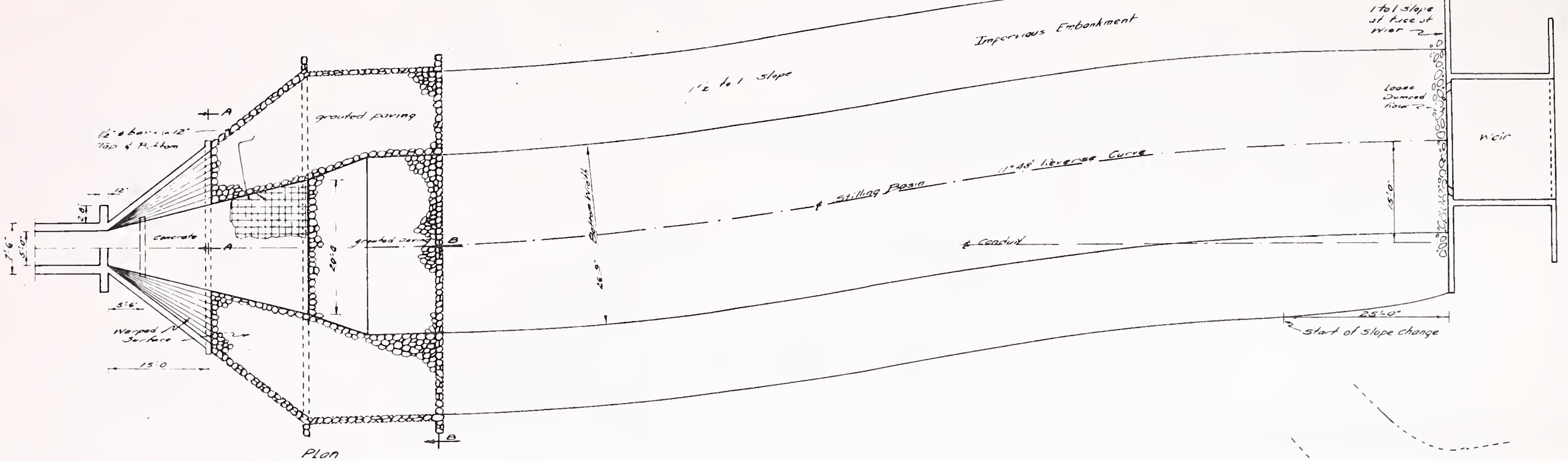
SPILLWAY PLAN & PROFILE
NEVADA CREEK STORAGE PROJECT
POWELL COUNTY

STATE WATER CONSERVATION BOARD

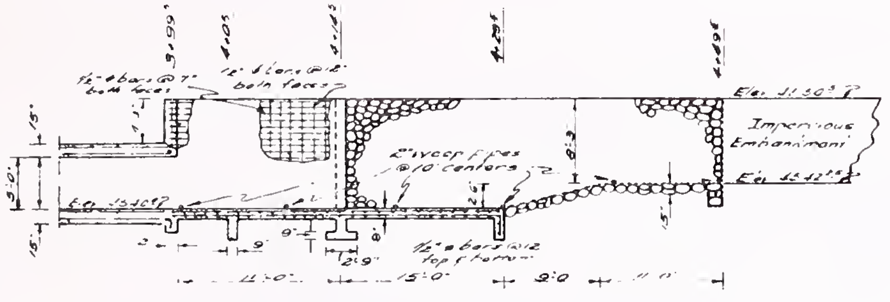
HELENA, MONTANA. DECEMBER 1937
SCALE AS SHOWN
P.W.A. NO. MONT. 1007-DS. DWG NO. 7

SHEET 3

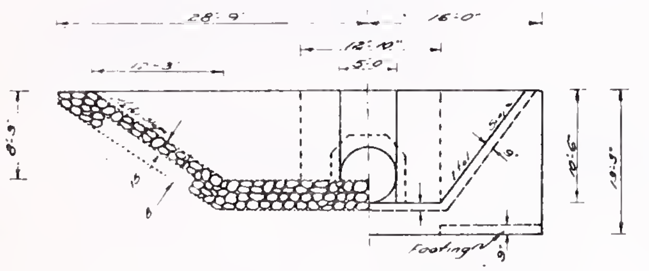
Revised by
Revised by



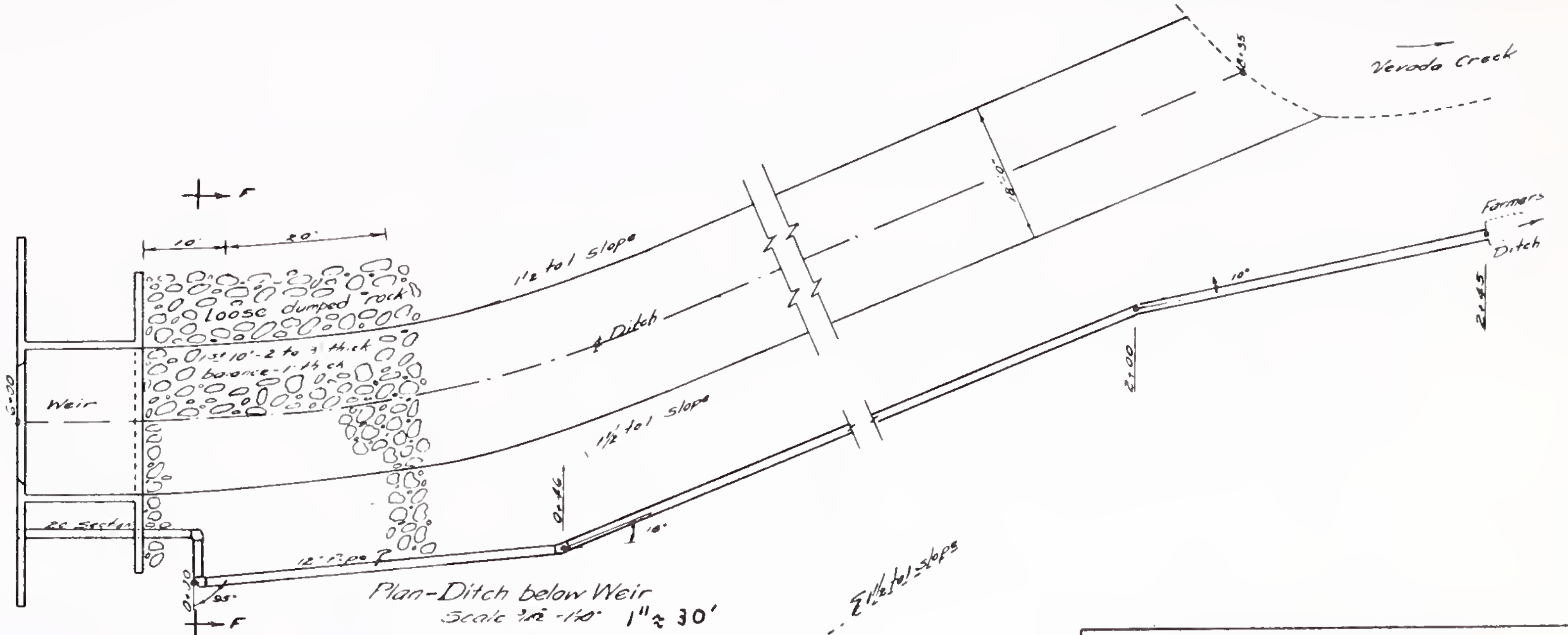
Plan
Outlet Structure & Stilling Basin
Scale ~~[illegible]~~ 1" ≈ 23'


½" ø bars @ 7" both faces
½" ø bars @ 12" both faces
Elev. 4130.5 ?
Impervious Embankment
Elev. 4542.5 ?
2" Weep Pipes @ 10' centers
Elev. 4540.0 ?
½" ø bars @ 12 top & bottom
15"
3'0"
8'3"
2'6"
9"
2'9"
11'0"
15'0"
9'0"

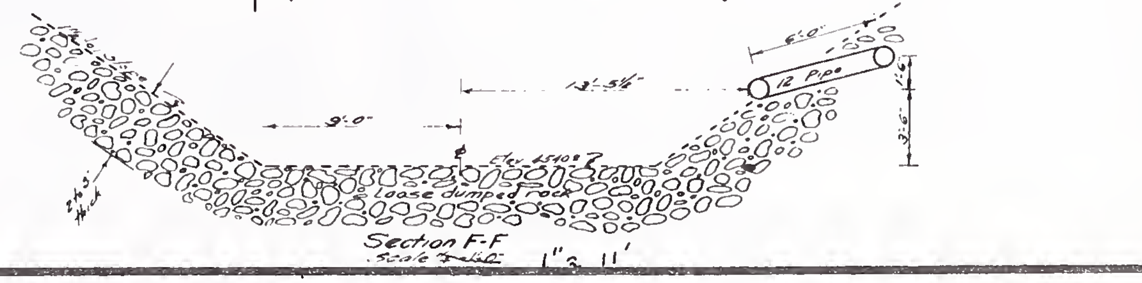
Elevation
Scale ~~[illegible]~~ 1" ≈ 23'



Section B-B Section A-A
Scale ~~[illegible]~~ 1" ≈ 23'



Plan-Ditch below Weir
Scale ~~[illegible]~~ 1" ≈ 30'

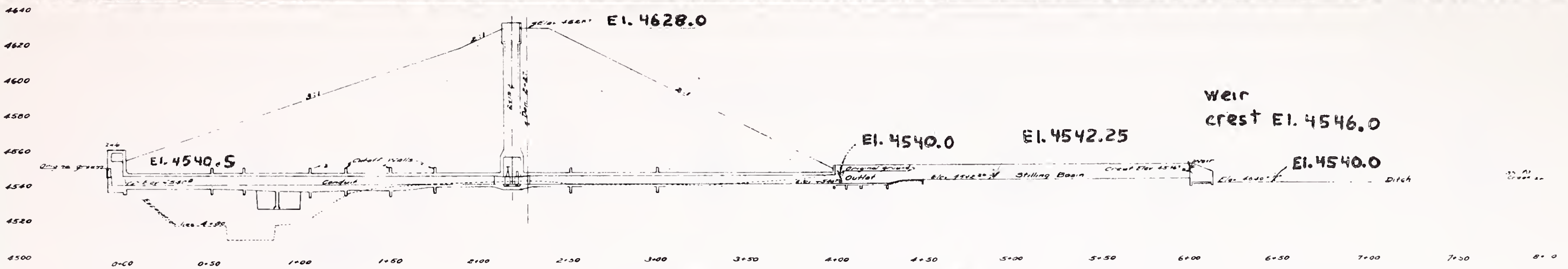


Section F-F
Scale ~~[illegible]~~ 1" ≈ 11'

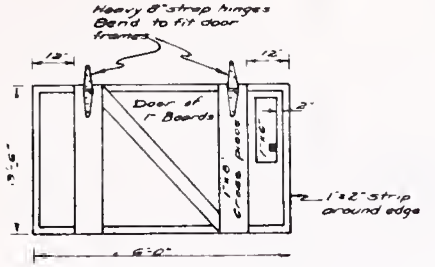
OUTLET STRUCTURES
NEVADA CREEK STORAGE PROJECT
POWELL COUNTY
STATE WATER CONSERVATION BOARD
HELMVILLE, MONT AUGUST 1938
SCALE AS SHOWN
PWA Nº MONT 1007 DS FIELD DWG Nº 21

Drawn by G.O.K.	Checked by RLS	
Traced by G.O.K.		

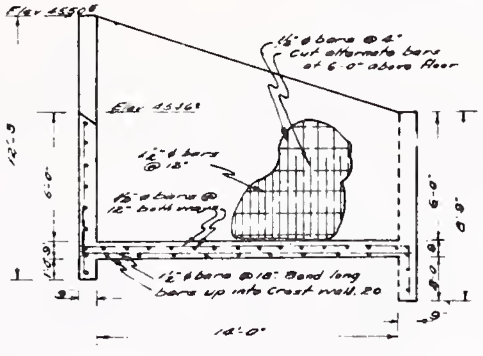
SHEET 4



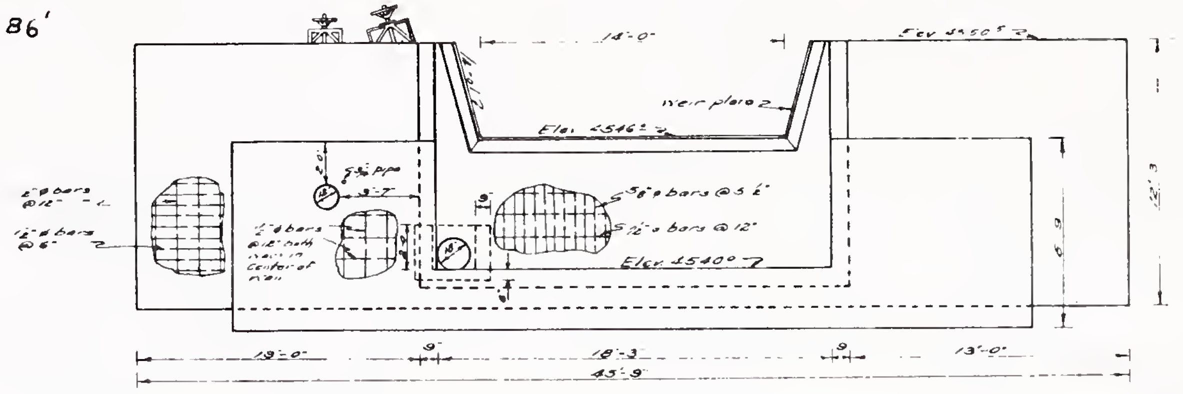
Profile
Scale 1"=30 86'



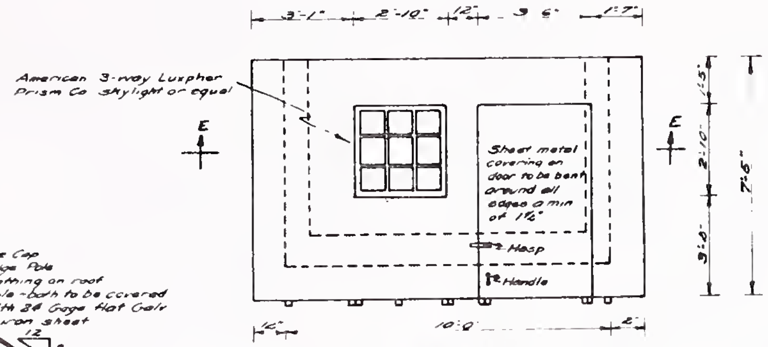
Plan of Bottom Side of Door
Scale ½"=1'-0" 1"≈6'


Elev 4550.0
Elev 4546.0
1½" ø bars @ 4" Cut alternate bars at 6'-0" above floor
½" ø bars @ 18"
½" ø bars @ 12" both ways
½" ø bars @ 18" Bend long bars up into Crest wall. 20
12'-9"
6'-0"
14'-0"
9"

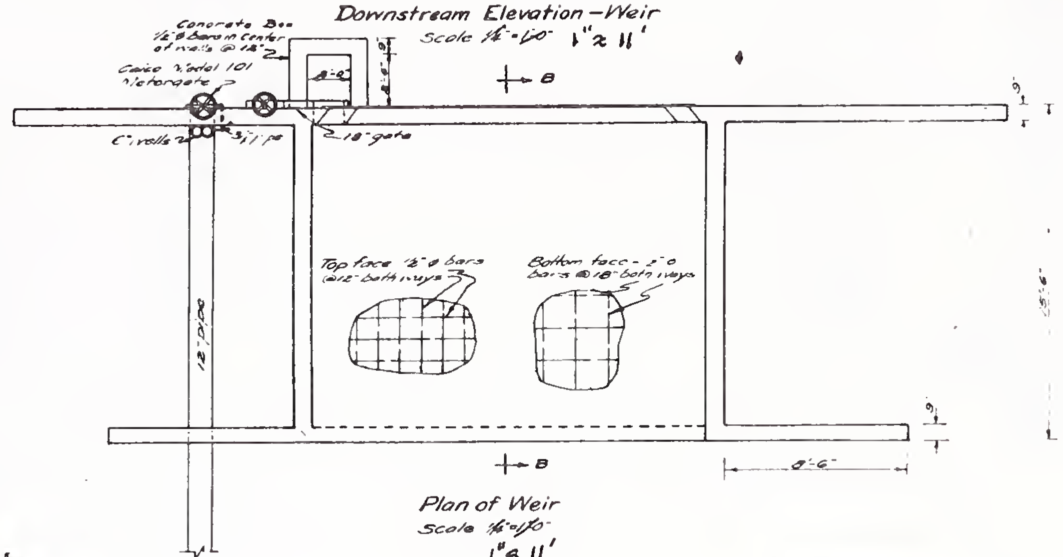
Section B-B
Scale ¼"=1'-0" 1"≈11'



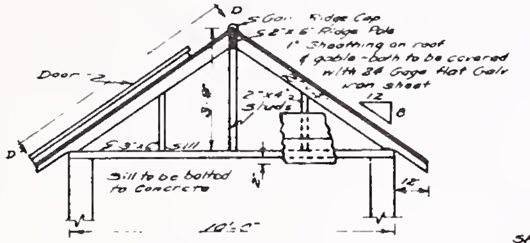
Downstream Elevation—Weir
Scale ¼"=1'-0" 1"≈11'


American 3-way Luxpher Prism Co skylight or equal
Sheet metal covering on door to be bent around all edges a min of 1½"
Hasp
Handle
E
E
3'-1"
2'-10"
12"
3'-6"
1'-7"
10'-0"
7'-6"

View D-D—Gate House
Scale ⅛"=1'-0" 1"≈8'

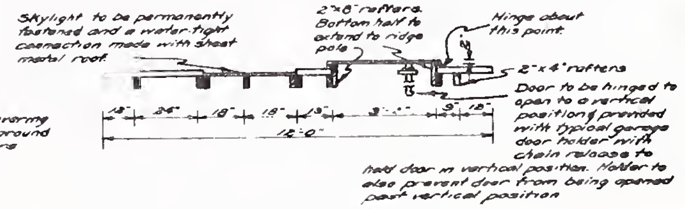

Concrete Box ½" ø bars in Center of walls @ 12"
Calco Model 101 Metergate
C. walls
18" gate
B
B
Top face ½" ø bars @ 12" both ways
Bottom face ½" ø bars @ 18" both ways
12" pipe
8'-6"

Plan of Weir
Scale ¼"=1'-0"
1"≈11'


D
D
Door
Galv Ridge Cap
2"x6" Ridge Pole
1" Sheathing on roof & gable - both to be covered with 24 Gage flat Galv iron sheet
2"x4" Studs
2"x4" Sill
Sill to be bolted to Concrete
12
8
10'-0"

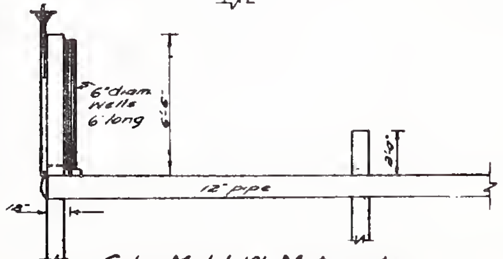
End View-Gate House Roof
Scale ⅛"=1'-0"
1"≈8'

Sheet metal roof covering to be bent up 2½" around door frame, & corners made water-tight


Skylight to be permanently fastened and a water-tight connection made with sheet metal roof
2"x8" rafters. Bottom half to extend to ridge pole
Hinge about this point
2"x4" rafters
12'-0"

Door to be hinged to open to a vertical position & provided with typical garage door holder with chain release to hold door in vertical position. Holder to also prevent door from being opened past vertical position

Section E-E—Gate House
Scale ⅛"=1'-0"
1"≈8'



Calco Model 101 Metergate
Scale ¼"=1'-0"
1"≈11'

GATE HOUSE & WEIR
NEVADA CREEK STORAGE PROJECT
POWELL COUNTY
STATE WATER CONSERVATION BOARD
HELMVILLE, MONT. AUGUST, 1936
SCALE AS SHOWN
P.W.A. № 1007 DS FIELD DWG № 22

Drawn by GAK	checked by RLS	
Traced by GAK		

SHEET 5

#6-112-25

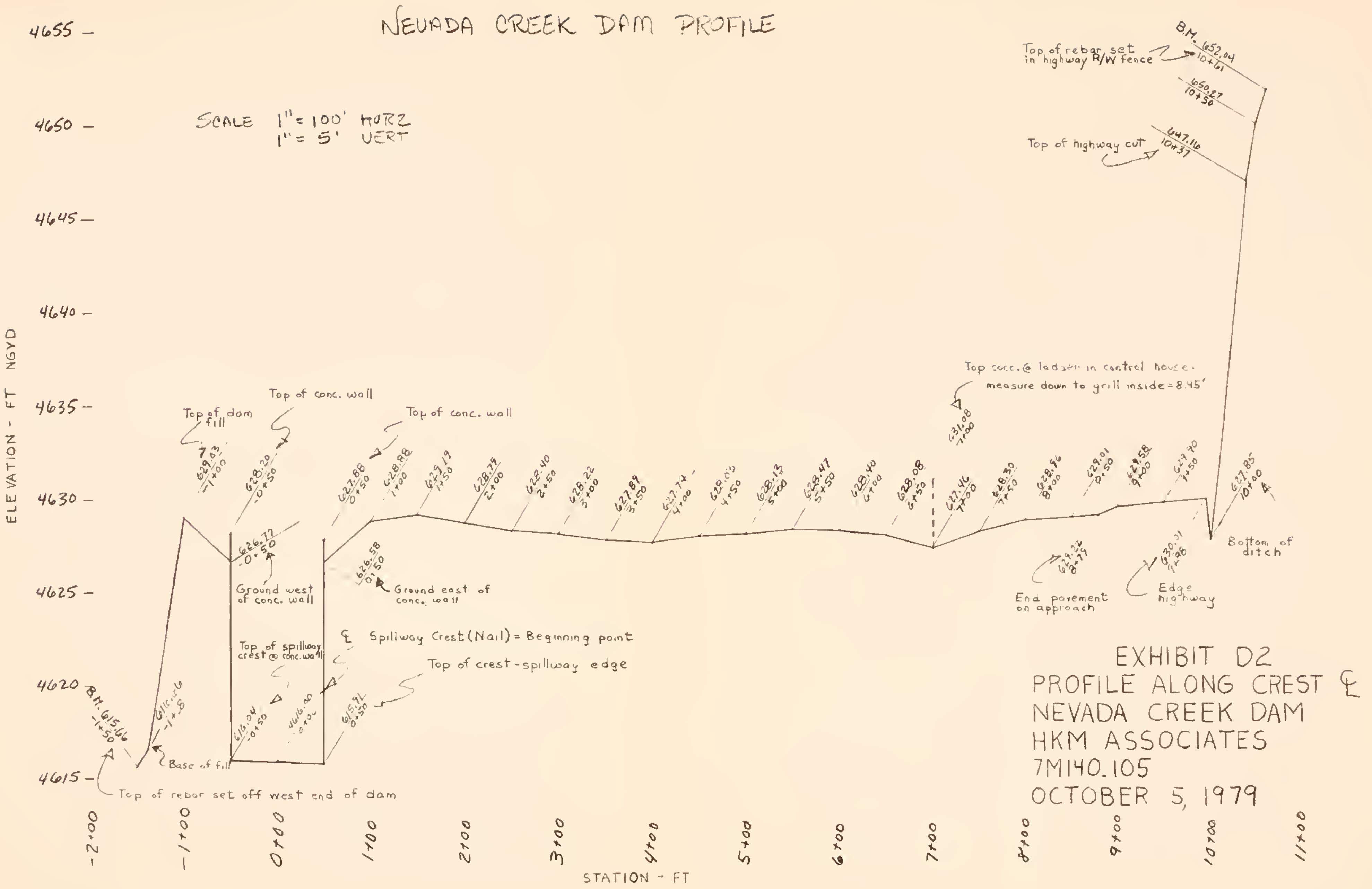


EXHIBIT D2
PROFILE ALONG CREST ℄
NEVADA CREEK DAM
HKM ASSOCIATES
7M140.105
OCTOBER 5, 1979

HKM ASSOCIATES COMPUTATION SHEET

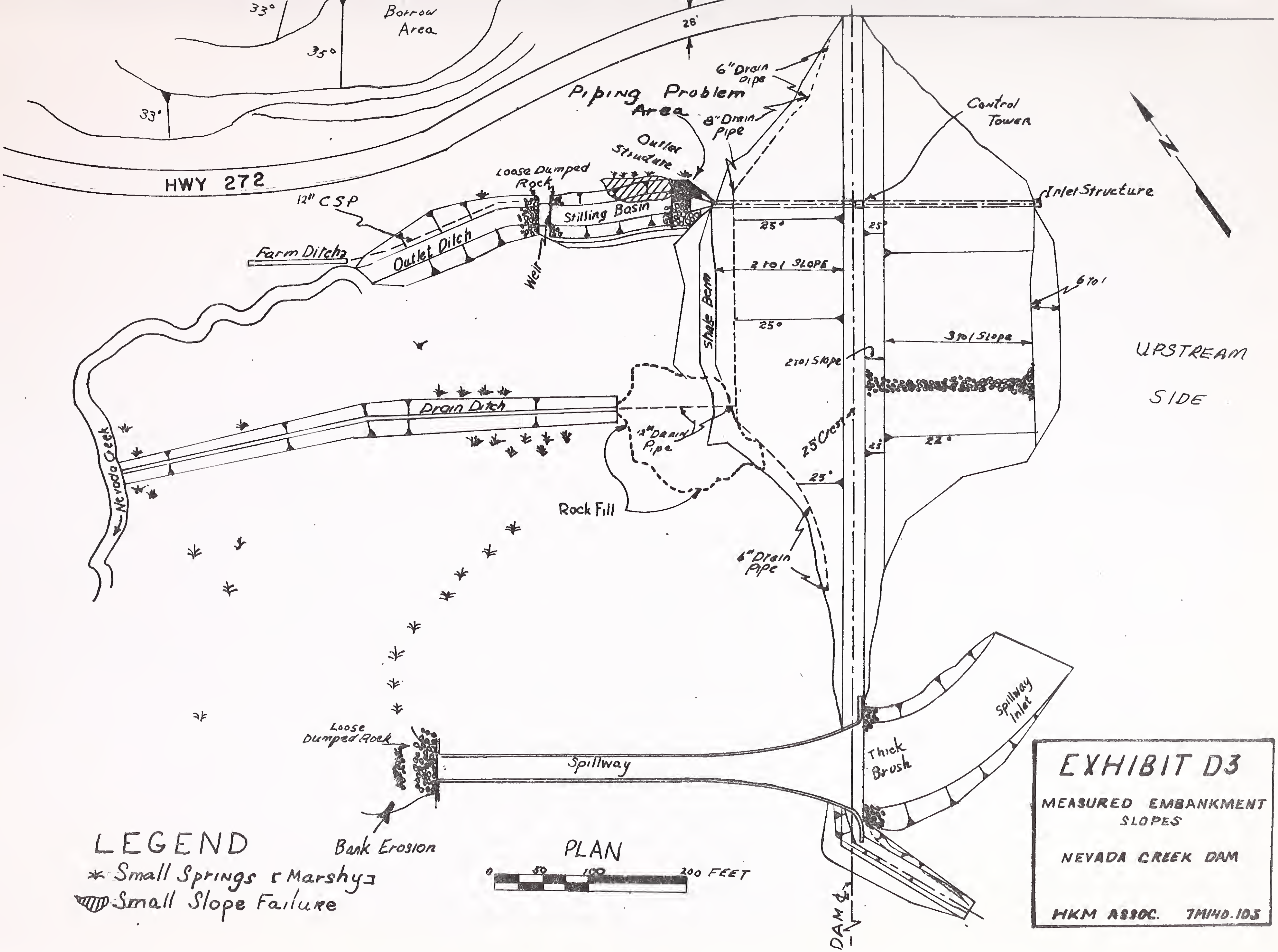
By MR	**Date** 10-5-79	**Project No.** 7M140.105	**Sheet** 1 **of** 2
Chkd by DRK	**Date** 10-15-79	**Name** NEVADA CREEK DAM CREST SURVEY	
Details		**Feature** EXHIBIT D2 STATION- ELEVATION TABULATION	

STATION ALONG DAM ℄	ELEVATION FT, NGVD	
-1+50	4615.66	B.M., Top of rebar off west end of dam
-1+38	4616.66	Base of dam
-1+00	4629.03	Top of dam fill
-0+50	4626.77	Ground west of conc. wall
-0+50	4628.20	Top of conc. wall
-0+50	4616.04	Top of spillway crest @ conc. wall
0+00	4616.00	℄ spillway crest, nail (Beginning point)
0+50	4615.91	Top of crest, spillway edge
0+50	4627.88	Top of conc. wall
1+00	4628.88	
1+50	4629.19	
2+00	4628.79	
2+50	4628.40	
3+00	4628.22	
3+50	4627.89	
4+00	4627.74	
4+50	4628.03	

HKM ASSOCIATES COMPUTATION SHEET

By MR	**Date** 10-5-79	**Project No.** 7M140.105	**Sheet** 2 **of** 2
Chkd by DRC	**Date** 10-15-79	**Name** NEVADA CREEK DAM CREST SURVEY	
Details	**Feature** EXHIBIT D2 STATION-ELEVATION TABULATION		

STATION ALONG DAM ℄	ELEVATION FT, NGVD	
5+00	4628.13	
5+50	4628.47	
6+00	4628.40	
6+50	4628.08	
7+00	4627.46	
7+00	4631.08	Top conc. @ ladder in control house
7+50	4628.30	
8+00	4628.76	
8+50	4629.01	
8+79	4629.22	End pavement on approach
9+00	4629.58	
9+50	4629.90	
9+88	4630.01	Edge of highway
10+00	4627.85	Bottom of ditch
10+37	4647.16	Top of highway cut
10+50	4650.27	
10+61	4652.04	B.M., Top of rebar set in highway R/W fence



PLAN

LEGEND
- * Small Springs [Marshy]
- ▨ Small Slope Failure

EXHIBIT D3

MEASURED EMBANKMENT SLOPES

NEVADA CREEK DAM

HKM ASSOC. 7M140.103

APPENDIX E

NEVADA CREEK DAM
ENGINEERING DATA

Exhibit E1	Reservoir Area - Capacity Table
Exhibit E2	Reservoir Area - Capacity Curve
Exhibit E3	Discharge Rating Table
Exhibit E4	Discharge Rating Curves

EXHIBIT E1

RESERVOIR AREA-CAPACITY TABLE

NEVADA LAKE

H.K.M. ASSOCIATES
PROJECT NO.:7M.140.1C
10/03/79
PAGE 2

ELEV FEET (NGVD)	CAP AREA 0	CAP AREA 1	CAP AREA 2	CAP AREA 3	CAP AREA 4	CAP AREA 5	CAP AREA 6	CAP AREA 7	CAP AREA 8	CAP AREA 9
4560.0	156 38	198 44	245 51	299 57	360 63	426 70	499 76	579 83	666 91	761 99
4570.0	864 107	975 116	1095 124	1224 133	1362 142	1508 151	1664 160	1829 170	2004 178	2185 185
4580.0	2373 191	2568 198	2770 205	2978 210	3190 214	3406 217	3625 221	3848 225	4074 229	4305 233
4590.0	4541 238	4781 243	5026 247	5275 252	5530 257	5790 262	6055 268	6325 273	6600 278	6880 283
4600.0	7166 287	7455 292	7750 297	8050 302	8355 307	8664 312	8979 317	9299 322	9624 327	9953 332
4610.0	10290 337	10630 342	10970 347	11320 352	11680 357	12040 362	12400 368	12770 373	13150 383	13540 394
4620.0	13940 405	14350 416	14770 427	15200 438	15640 449	16100 460	16560 472	17040 484	17530 495	18030 507

NOTES : AREA IN ACRES
CAPACITY IN ACRE-FEET
CALCULATION BY CONIC METHOD
Source: DNRC Files
Capacity reduced to reflect sedimentation up to Aug., 1973

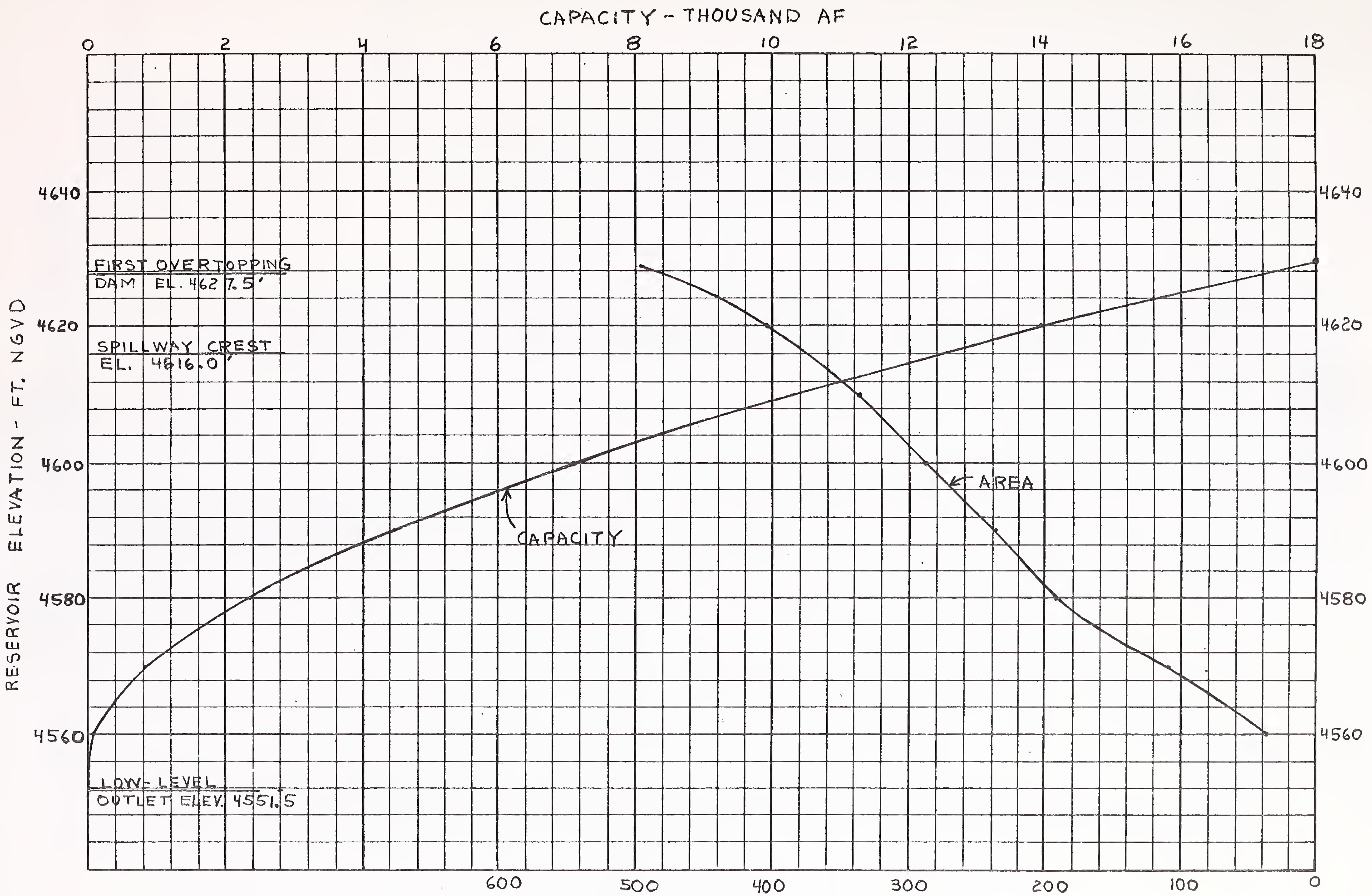


EXHIBIT E2
RESERVOIR AREA-CAPACITY CURVE
NEVADA CREEK DAM

EXHIBIT E3
DISCHARGE RATING TABLE
NEVADA CREEK DAM

	ELEVATION (FT. NGVD)	OUTLET WORK DISCHARGE (CFS)	SPILLWAY DISCHARGE * (CFS)	TOTAL DISCHARGE (CFS)
	4551.0	0.		0.
	4553.0	151.		151.
	4555.0	213.		213.
	4557.0	261.		261.
	4559.0	302.		302.
	4561.0	337.		337.
	4563.0	370.		370.
	4565.0	399.		399.
	4567.0	427.		427.
	4569.0	453.		453.
	4571.0	477.		477.
	4573.0	500.		500.
	4575.0	523.		523.
	4577.0	544.		544.
	4579.0	565.		565.
	4581.0	584.		584.
	4583.0	604.		604.
	4585.0	622.		622.
	4587.0	640.		640.
	4589.0	658.		658.
	4591.0	675.		675.
	4593.0	691.		691.
	4595.0	708.		708.
	4597.0	724.		724.
	4599.0	739.		739.
	4601.0	754.		754.
	4603.0	769.		769.
	4605.0	784.		784.
	4607.0	798.		798.
	4609.0	813.		813.
	4611.0	826.		826.
	4613.0	840.		840.
	4615.0	854.		854.
Spillway Crest	4616.0	860.	0.	860.
	4617.0	867.	320.	1187.
	4618.0	873.	900.	1773.
	4619.0	880.	1600.	2480.
	4620.0	886.	2600.	3486.
	4621.0	893.	3750.	4643.
	4622.0	899.	5000.	5899.
	4623.0	905.	6400.	7305.
	4624.0	912.	8000.	8912.
	4625.0	918.	9700.	10618.
	4626.0	924.	11600.	12524.
	4627.0	930.	13500.	14430.
First Overtopping	4627.5	933.	14400.	15333.
	4628.0	936.	15450.	16386.

* Spillway discharge based on existing conditions of brush cover in the approach channel

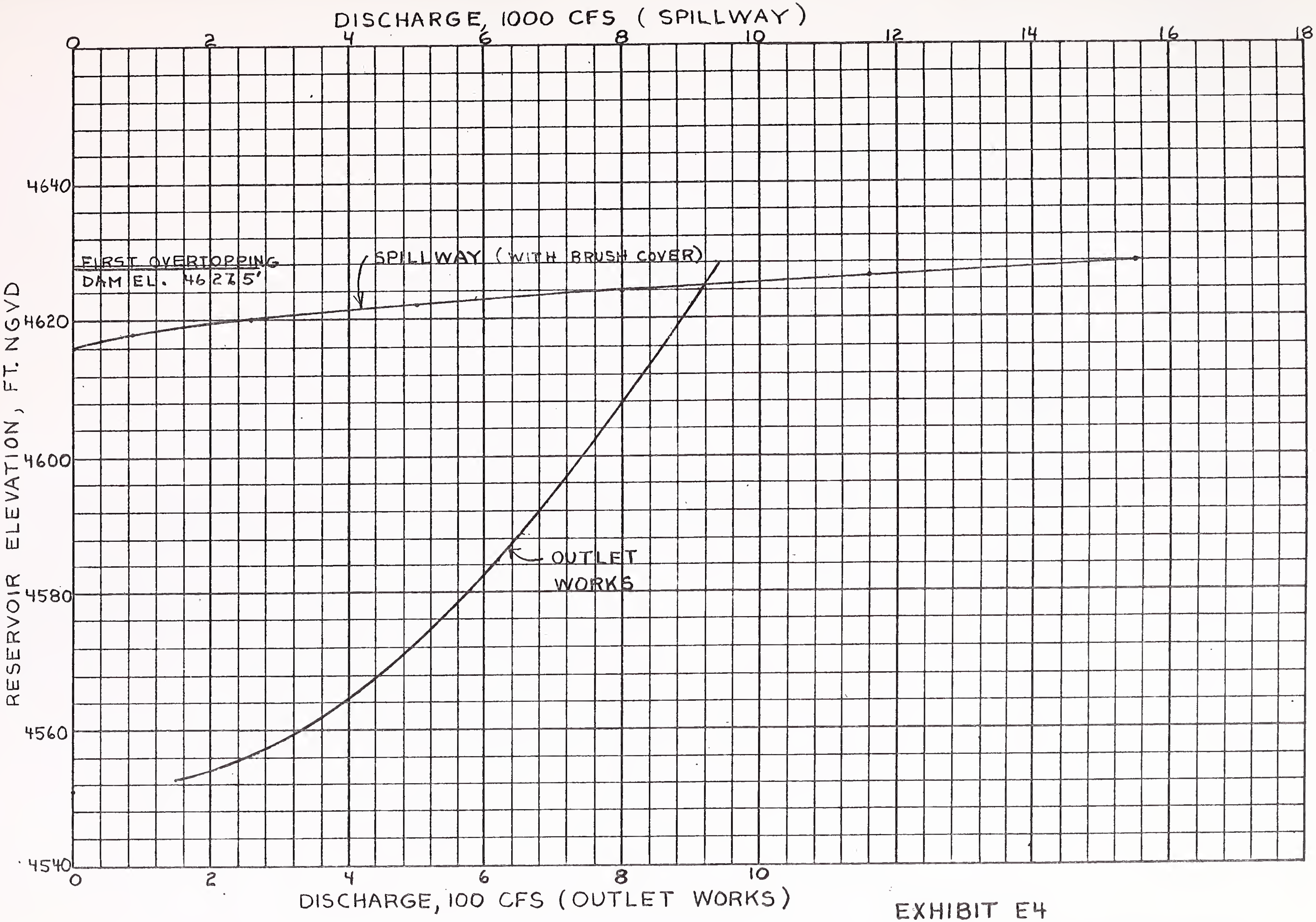


EXHIBIT E4
DISCHARGE RATING CURVE
NEVADA CREEK DAM

APPENDIX F

NEVADA CREEK DAM

CORRESPONDENCE

DEPARTMENT OF NATURAL RESOURCES
AND CONSERVATION
WATER RESOURCES DIVISION

THOMAS L. JUDGE, GOVERNOR
32 SOUTH EWING
STATE OF MONTANA
(406) 449-2872
HELENA, MONTANA 59601

December 12, 1980

Department of The Army
Seattle District, Corps of Engineers
P.O. Box C-3755
Seattle, WA 98124

Attn: Ralph Morrison

Re: HKM Dam Safety Inspection Report on Nevada Creek Dam (MT-17)

Dear Ralph:

The Department of Natural Resources and Conservation has reveiwed the above referenced final draft report. We concur with the findings and recommendations, and find that it satisfies the criteria of the Phase I report. The majority of comments are editorial and have been previously discussed with staff.

One comment we feel is significant enough to be pointed out in this letter, relates to the repairs that were completed in the spring of 1974. Although I believe no documentation was provided to HKM to verify this, the jet pump was repaired or replaced under the same maintenance program which refurbished the operating gate, covered the emergency gate with a protective epoxy coating and the conduit metal lining with a rust and wear resistant material.

Also, in September of 1980, the Nevada Creek Water Users completed a concrete overlay to the spillway floor. A memo on this repair work is inclosed.

Two copies of the final draft report were sent to the Nevada Creek Water Users Association but they chose not to comment.

Thank you for the opportunity to review and comment on this final draft report on Nevada Creek Dam.

Sincerely,

Richard L Bondy

Richard L. Bondy, P.E.
Chief, Engineering Bureau
(406) 449-2864

"AN EQUAL OPPORTUNITY EMPLOYER"

MEMORANDUM

TO: Rick Bondy, Chief
Engineering Bureau

FROM: Robert Clark, Supervisor Robert Clark
Design Section
Engineering Bureau

SUBJECT: New Overlay on Nevada Creek Dam Spillway

I observed the new construction being done on the spillway on September 17, 1980. The contractor was Lowen & Van Dehey from Missoula. Monroc was the concrete supplier, also from Missoula. The concrete being used was a 7-sack mix design for a 5,000 psi break strength. The new concrete was being bonded to the existing spillway floor with an epoxy bonding agent. The epoxy was Dow DPS Modifier H.

Two by fours (2 x 4), on edge, were being used for forms. The existing floor was being cleaned of all loose material prior to placement of the new concrete. No steel reinforcing was being installed. The construction joints were butt joints with no provisions made for expansion and contraction. No water stops were put in. The edges of the butt joints were being bevelled about 1/8 inch wide by 1/2 inch deep.

Once the bevelling was completed, the construction joints were to be covered by a tar type sealer. After the overlay had cured for 14 days and epoxy sealer was to be spread on the new concrete, which would seal it.

The new overlay ended at the break in slope of the spillway floor. (STA 10+00.) The transition from the new slab to the existing slab ended in an abrubt drop. (STA 10+00 to STA 10+9.43).

The transition reduced the thickness of the new slab from 3 1/2 to 1 1/2 inches. During high spillway flows, this 1 1/2 inch drop could be a potential source for cavitation. The new slab is 381' 7" long measured from the spillway crest.

Three cylinders for testing were cast and a slump test taken. The slump was between 6 1/2 to 7 inches.

On September 23, 1980, I cast three more test cylinders and took another slump test. The slump was 5 1/2 inches. I told the contractor I felt they were using to much water; but they disagreed. Shrinkage cracks were evident in sections of the new overlay that were partially cured.

I advised them to put weep holes in the sidewalls every 10 feet, but later they told me they were only going to put in a total of 60 weepholes which would be furhter than 10 feet apart.

On October 1, 1980, I observed the last section placed. The first truck of concrete sat for one hour and forty five minutes before it was placed. The section over the crest was placed before remainder of the floor section and was poured from the second truck load of concrete.

The new overlay started 4 feet down the front face of the new spillway crest. More water was added to the mix before the floor section was poured. I would estimate a slump of about 8 inches. The recommended water-cement ratio for a placement of this kind should yeild a slump of 3 or 4 inches.

Numerous shrinkage cracks are already present in the new overlay. Some cracks are as much as 3/32 inch wide and run the length and width of the new slabs.

The bond between slabs is questionable due to the concrete shrinkage and to the condition of the existing concrete. Some method of mechanically bonding the new overlay to the existing spillway floor would be preferable.

The first cylinder break from the cylinders cast on September 17 and 22, 1980, was 2,500 psi and 2,680 psi respectively. The low strengths indicate the concrete will never reach the designed 5,000 psi strength.

I do not feel this has been a quality job and that the servicibility of the new slab will not be very long because of following reasons:

1. The new slab was not mechanically attached to the existing slab.

2. No water stops were put in the joints.

3. No expansion and contraction joints were installed to allow the slabs to move without building high stresses which can crack and break the new overlay.

4. No quality control on the placement of the concrete.

5. The new slab was cracking with long pattern and longitudinal cracks from the shrinkage of the concrete.

6. The shrinkage could affect the bond between the existing slab and the new slab.

7. Water will seep through the shrinkage cracks and freeze, which will break up the new slab.

8. Even though an epoxy is placed over the new slab which may seal the existing cracks, new cracks will form as the concrete continues to cure and shrink.

9. Putting the overlay on and trying to bond it to the existing slab with epoxy may not work because of the poor quality of concrete beneath the new overlay.